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And
WASTEWATER ASSOCIATION

REGINA, SASKATCHEWAN
October 17, 18, 19, 1990

WESTERN CANADA WATER AND WASTEWATER ASSOCIATION

WESTERN CANADA SECTION A.W.W.A.

WESTERN CANADA POLLUTION CONTROL ASSOC. W.P.C.F.

PROCEEDINGS

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WESTERN CANADA WATER AND WASTEWATER ASSOCIATION

including the

20th ANNUAL CONVENTION
WESTERN CANADA SECTION
AMERICAN WATER WORKS ASSOCIATION

and the

18th ANNUAL CONVENTION
WESTERN CANADA POLLUTION CONTROL ASSOCIATION
WATER POLLUTION CONTROL FEDERATION

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WESTERN CANADA WATER AND WASTEWATER ASSOCIATE MEMBER LISTING

CONTROLLING INDUSTRIAL DISCHARGES TO SEWERS

Rob Sinclair and Roy Houston

KGS GROUP

Introduction

The Municipal Industrial Strategy for Abatement (MISA) Program is an initiative developed by the Ontario Ministry of the Environment (MOE). This program is directed at the minimizing of toxic contaminants into mainly the aquatic environment. The focus of the program is on municipal and industrial discharges, as presented in **Figure 1.**

In Ontario, the industrial sectors comprise approximately two thirds of the 300 direct dischargers. The remaining dischargers are covered in other minor sectors. Besides the direct dischargers, a number of toxic wastes are discharged to wastewater collection systems. These dischargers will be controlled using municipal abatement programs.

The focus of this paper is on the MISA abatement strategy being evaluated by the MOE as part of a 5 municipality demonstration study. The objective of the demonstration program is threefold, firstly to evaluate the procedures developed to minimize toxic discharges into these community based wastewater collection and treatment systems and ultimately into the local aquatic environment; secondly, to protect workers from toxic gases and vapours within the collection and treatment system; and thirdly, to protect the treatment facility from upsets.

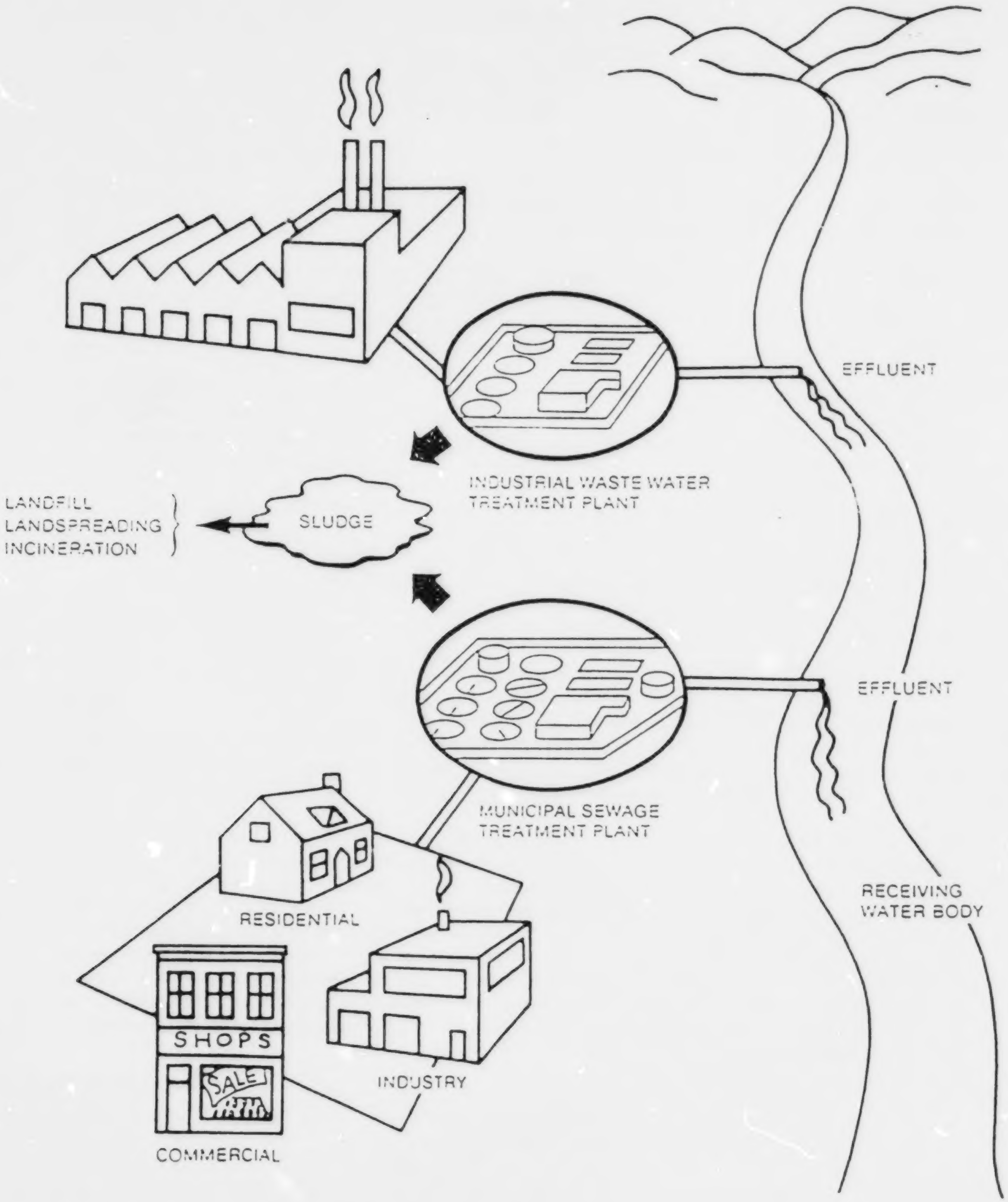
The KGS Group has been retained by the MOE and the City of Thunder Bay to provide technical assistance through all components of the demonstration project in Thunder Bay.

The MISA Program

The Misa Program was implemented to address the increasing number of environmental concerns associated with water quality in Ontario. For example, mercury and dioxin have effected water quality and are or have been accumulating in the food chain as observed in fish quality evaluations in several areas of the province. In the case of mercury, health problems have been directly linked to their uncontrolled discharges. Within the Great Lakes system, over 800 different chemical components have been isolated.

The ability of present wastewater treatment programs to cope with this problem has been hindered by outdated control concepts. The present concepts have been directed at the conventional pollutants including biodegradable wastes, suspended solids, ammonia, phosphorous, some metals, phenols, oils and grease. The MISA program will still maintain the reduction of conventional pollutants, as a cornerstone of the overall strategy, but will add new methodologies to deal specifically with toxic contaminants. The contaminants of major concern are the toxic heavy metals (mercury, arsenic, lead, etc.) and toxic organics (carbon based compounds, PCB's, dioxins and chlorinated benzene). To reduce these toxic dischargers a 10 point Sewer-Use Control Plan has been developed. The 10 Point Plan is outlined in the following section with reference to the Thunder Bay situation.

FIGURE 1: POLLUTION PATHWAYS FOR MUNICIPAL/INDUSTRIAL WASTE WATERS


EFFLUENT
INDUSTRIAL WASTE WATER TREATMENT PLANT
LANDFILL
LANDSPREADING
INCINERATION
SLUDGE
EFFLUENT
MUNICIPAL SEWAGE TREATMENT PLANT
RESIDENTIAL
RECEIVING WATER BODY
SHOPS
SALE
INDUSTRY
COMMERCIAL

THE 10 POINT SEWER-USE CONTROL PLAN

Development of Public Participation Program

Throughout the development of the MISA program, there has been public involvement. This involvement has resulted in many contributions to the overall program and to the acceptance of strategies that are to be implemented. It is important both technically and politically that the Public be given an opportunity to be involved in all aspects of the 10 Point Plan. In Thunder Bay, newsletters have been forwarded to special interest groups and newspaper ads have been used to inform the general public of public meetings. Initial public meetings have not been well attended, however, it is anticipated that the interest level will improve for the full MISA program, as public awareness increases.

Legal Authority

The MOE has investigated and developed the necessary legal mandates at both the provincial and municipal level to implement the program. The legal implementation has been addressed through the Municipal, Provincial Offences, Ontario Water Resources and Environmental Protection Acts. A Model Sewer-Use Bylaw was developed for each municipality or city to use as an initial guideline document. This model Sewer-Use Bylaw will be modified on an ongoing basis to set local discharge and statute requirements. The Sewer-Use Bylaw forms the basis for developing legally enforceable discharge agreements with industrial dischargers.

Industrial Waste Survey

The industrial waste survey is an important initial phase of the technical data evaluation that is directed at the compilation and classification of all non-domestic dischargers to form a comprehensive master list. In Thunder Bay the master list was composed of 5200 non-domestic dischargers. These dischargers were classified using the Standard Industrial Classification [SIC] system and then inputed into a computer program which was designed for data manipulation. This program, coupled with mail-out documents requesting wastewater information, provided an effective technique to focus on the major industrial dischargers. The basic waste survey process is presented in the flow chart of **Figure 2.**

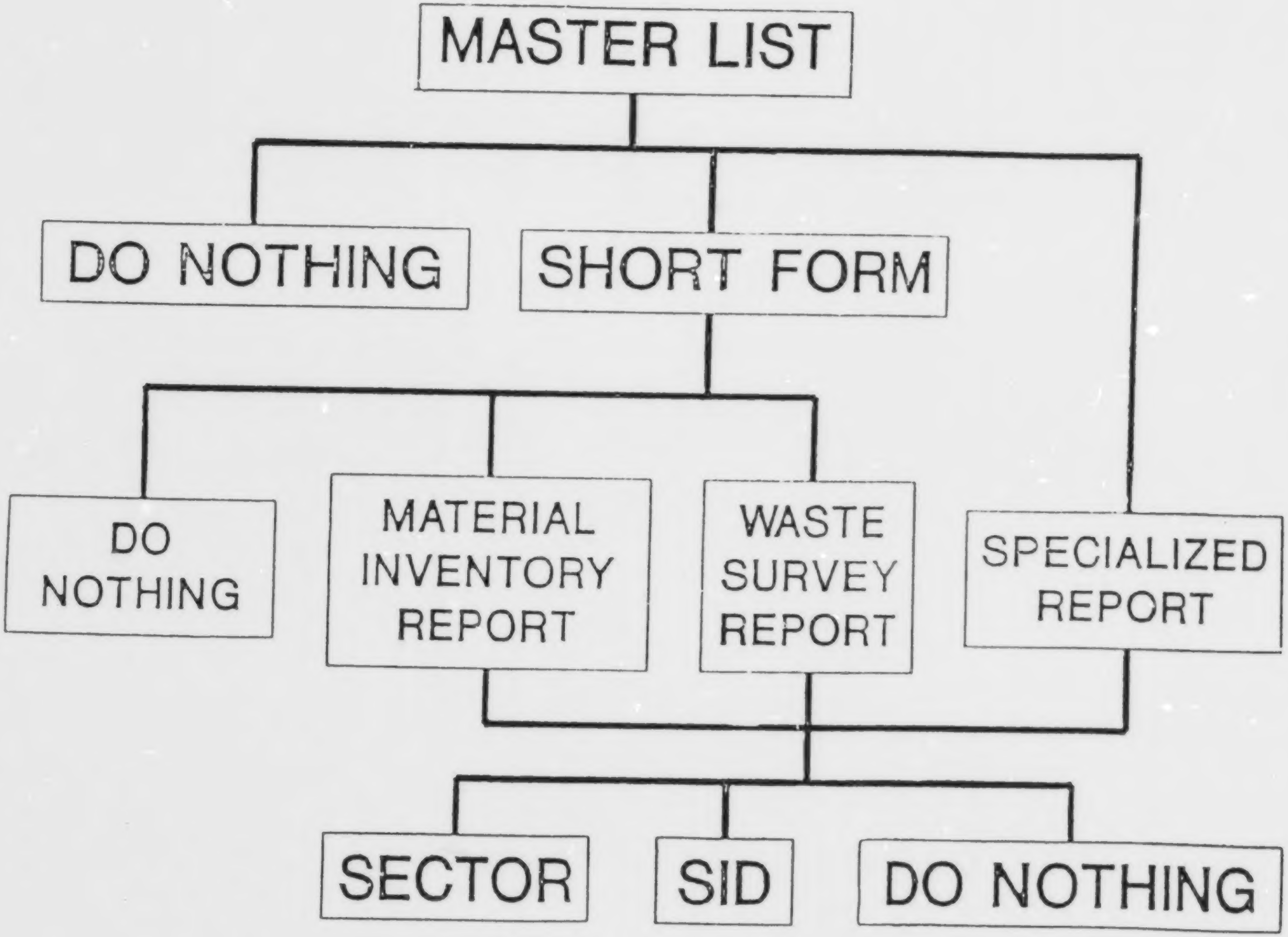
Compilation of Technical Data and Sampling Program

This component is very important, as it forms the data base for all subsequent components of the study. All background data from domestic water quality, through to receiving water quality, must be compiled and evaluated. This information is used to define the scope of sampling programs for the sewage treatment plant [STP] and in-sewer sampling program. This data further facilitates the development of the specific sampling program required for each major discharger.

Development of Sewer-Use Limits

The development of local limits is necessary for each municipality to ensure that the individual sewage collection and treatment systems, as well as local receiving waters, have site specific discharge control limits. There are a number of approaches or methodologies applicable to the development of local sewer-use limits, as outlined below.

- Allowable Headworks Loading Method
- Collection System Approach
- Industrial Use Management Practice Plans
- Case by Case Permitting


MASTER LIST
DO NOTHING
SHORT FORM
DO NOTHING
MATERIAL INVENTORY REPORT
WASTE SURVEY REPORT
SPECIALIZED REPORT
SECTOR
SID
DO NOTHING

The Allowable Headwork Loading Method was used in the MISA program, as this method facilitated the development of specific numeric local limits. This procedure, in conjunction with specific components of the other methods, provides for a comprehensive local limits package within the collection system and through the STP to the receiving body.

The allowable headwork loading method evaluates the local limit for 4 different scenarios, namely:

1) Pass-through affecting receiving water

2) Inhibition of the STP Process

3) Inhibition of the Sludge Digestion Process

4) Degradation of Digestion Sludge Quality

The most limiting value for each parameter, evaluated for the above four scenarios, is designated as the local limit.

The Thunder Bay experience has demonstrated some difficulties with the overall process. For example, aquatic organisms are quite sensitive to copper. However, with significant amounts of copper being leached from domestic plumbing systems, the local limits process indicates that there should be no copper discharges allowed by industries. It is totally impractical to eliminate all copper discharges by industries, such that some general procedures will be necessary to adequately handle such situations.

Development of SID Sampling and Inspection Program

The focus of this component is the development of a comprehensive monitoring program which includes both a self-monitoring strategy and audit sampling and inspection strategies. The program is designed to work closely with industry, as much as possible, but will have the flexibility to evaluate discharges independently. An integral part of this component is the assessment of manpower, as well as field and laboratory equipment requirements.

Development of Control Instruments and Enforcement Strategy

A major objective of this component is to characterize additional discharges to the STP from such sources as:

- hauled sewage or industrial waste

- landfill leachate

- cooling waters

- accidental spills

Industries with potential for accidental spills to enter the sewer system, may require an Industrial Discharge Management Plan (IDMP).

A second objective of this component is the identification and classification of the various forms of non compliance and the coupling of this with a specific tracking system and practical enforcement response.

Development of Sewer-Use By-Law

The updating of the Sewer-Use By-law is ongoing with regular reviews as the data base increases. The initial by-law is developed from components outlined in the previous section. For example, the Local Limits are incorporated into the by-law and form a basis for limiting toxic discharges by industry.

Development of Effluent Discharge Permits for SID's

Once the Sewer Use By-law is in place , a permitting system is required to control the discharges to Significant Industrial Dischargers (SID's). The permits outline the quantity and quality of discharges allowed within specific limits as well as the requirements for self-monitoring and accidental discharge reporting.

Evaluation of Program Resource Requirements and Financing Options

The focus of this step is the determination of overall resource requirements with their associated costs and balance these costs against the revenue developed by practical user-pay options. The preliminary indications for Thunder Bay indicated that the major removal will be developed by the conventional waste (suspended solids, phosphorus) mainly from the food industry.

Sewer Use Control Program For The Prairies

The MISA program is mandated and funded by the Ontario Ministry of the Environment to ensure adoption by all major communities in Ontario. At present, there is no similar initiative at the provincial level on the Prairies. However, there are still opportunities for Prairie communities to develop programs focused at toxic waste discharge reduction to their local environment. Practical alternatives to the MISA program include:

- a more focused scope to suit local conditions
- the formation of district user groups
- larger centres providing regional service to small communities on a fee for service basis.

The development of sewer-use control programs using any of the above options would require a co-operative effort of interested communities, provincial environmental departments, as well as engineering specialist familiar with the development of the MISA or similar initiatives.

In many cases, most of the resources necessary to address the major components of a sewer-use control program are already in place. These communities require only technical expertise to develop a program focused at their specific needs. The starting point could be as simple as a one week sampling program at the STP to determine if there are any levels of toxic contaminants to be concerned with. A program of this type could be initiated for only a few thousand dollars.

ACKNOWLEDGEMENT

We would like to acknowledge the contribution and the participation of the City of Thunder Bay, Ontario, and the Ontario Ministry of the Environment staff (in the project) and would like to thank them for providing the background for this paper.

MUNICIPAL WATER SUPPLIES - ROLE FOR DEMAND MODIFICATION MEASURES?

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INTRODUCTION

Population growth and lifestyle changes are increasing demands on existing water supplies. For most communities, new water supplies are becoming scarcer and more costly to develop. As communities begin to explore additional water supply alternatives, water conservation is becoming more prevalent in North America as a means of meeting the growing needs of many communities.

Conservation serves three distinct roles in water supply planning.

- One role is as a drought response measure, to be implemented when traditional water supplies are temporarily limited. Some feel this is the appropriate role for conservation because if allows maximum flexibility in operating the water supply system. In reality, this approach creates flexibility because it develops more water supplies than might be the case if long-term conservation were practised.
- The second role is as a long-term demand modification measure, essentially acting as an additional water supply.
- The third is to reduce peak demands. While most conservation measures help reduce peak demand, measures are specifically targeted to shifting the time of day when water is used.

Regardless of the role conservation is used for, selecting and implementing a water conservation program is the task of the community's policy makers. In planning a conservation program, they need to balance several issues, including which water supplies are implemented, whether the conservation program creates any social or economic inequities, and how program improvements will be financed.

Planning a water conservation program should be a cumulative process to allow public participation and input throughout. By soliciting public involvement, policy makers can get a better reading on the level of lifestyle changes that might be acceptable under a potential conservation program.

The typical steps in preparing a conservation program for long-term savings are:

- Project water demand under existing conditions.
- Determine the role of conservation.
- Estimate conservation savings from potential water conservation measures.
- Develop alternative conservation programs.
- compare conservation programs to other water supply alternatives.
- Select a conservation program consistent with a water supply plan.
- Begin implementing the plan.

The steps for designing a drought response program are similar except that drought response programs do not need to be compared to alternative water supplies. These steps are described in more detail below. [Some of these steps are also described in the American Water Works

Association publication, Water conservation.)

PROJECTING WATER DEMAND UNDER EXISTING CONDITIONS

There are varying levels of sophistication in projecting water demand. The simplest form of projection is extrapolating historical water demands. Several methods are based on multiplying population forecasts by a per capita water use. The methods vary by the way the per capita use is calculated (historical average, regression analysis, etc.). The most detailed method for estimating water demands is use of a model developed by the U.S. Army Corps of Engineers, Institute of Water Resources. This model, called IWR-MAIN, is a disaggregated water demand forecast model. (For a more detailed description of the IWR-MAIN model, see Krouse, 1990).

Regardless of the method used, in order to analyze water conservation, demand should be disaggregated into user classes or sectors. At a minimum, two sectors should be identified, residential and commercial/industrial. The conservation program can be more focused and analysis more refined with a greater disaggregation. Some of this information may be obtained from the water billing system and/or water rate study.

DETERMINING ROLE OF CONSERVATION

With an understanding of the projected water demand, potential water supplies, and the water distribution system, the community leaders need to define the role of conservation (long-term water supply, drought response, or peak reduction). The role of conservation will play a large part in determining what types of measures are appropriate for inclusion in the conservation program. Table 1 lists some of the more common measures for each role of conservation. Several of these measures serve more than one role.

Table 1
Typical Conservation Measures by Role

Long-Term Water Supply	Drought Response	Peak Reduction
Low-flow Fixtures	Retro-fit Kits, Limited	Water Pressure Reduction
Low-flow Appliances	Seasonal Rates	Water Meters, Pricing
Retro-fit Kit, Permanent	Commercial Water Audits	Seasonal Rates
Water Meters, Pricing	Irrigation Advisor	Irrigation Audits
Lawn-watering Guides	Irrigation Audits	Xeriscape
Xeriscape	Watering Restrictions	Water Education
Water Education	Water Education	

ESTIMATING WATER CONSERVATION SAVINGS BY MEASURE

Water conservation measures are the building blocks of a conservation program. Measures are designed to influence specific user groups; they are then grouped into conservation programs that are tailored to the goals and needs of the community.

Water conservation measures can be most easily discussed if they are categorized into structural versus nonstructural measures and voluntary versus mandatory measures. These categories gain importance when considering the role of conservation. Structural measures are those that produce savings once they are installed and continue to generate predictable savings. Examples include low-flow fixtures or landscaping restrictions. Savings from structural measures take some time to implement.

Nonstructural measures produce variable savings but results are more immediate. Nonstructural measures include water rate changes and public education/awareness programs. These measures need to be monitored closely to maintain their effectiveness.

Voluntary measures are designed to let the customer decide about implementing the measure. These programs have few political repercussions but may cause some social inequities for those affected by the measure. An example of this would be implementing an inverted rate structure instead of a mandatory cutback of lawn watering. Low-income families might see their water bill rise a significant amount while higher income families continue to use the same amount of water with an insignificant change in their disposable income.

Mandatory measures are imposed on the public. These programs are less desirable to the average consumer, but they can create immediate savings and cover all income groups. Typical mandatory measures include lawn-watering restrictions, plumbing code changes, and landscape ordinances.

For long-term conservation savings, the most effective measures are structural and primarily voluntary. Drought response measures are typically nonstructural but mandatory. Peak-demand-reducing measures are usually nonstructural and voluntary.

Several methods can be used to estimate water savings. Spreadsheet models based on savings estimates in the 1984 HUD studies can give a preliminary estimate of savings. The California Department of Water Resources had developed a computer model to estimate water conservation savings. This program, WaterPlan^{TM}, is used throughout California to prepare urban water management plans (California, 1989). WaterPlan^{TM} has been developed solely to measure conservation savings, so water demand patterns must be determined before the model is run. The most sophisticated program available is the IWR-MAIN model. This model, which contains a wealth of information in its default parameters, requires significant data for calibration.

DEVELOPING ALTERNATIVE CONSERVATION PROGRAMS

The number of conservation measures that are considered can be cumbersome so the most efficient way of evaluating conservation is by assembling measures into programs that reflect degrees of implementation difficulty. For example, a base program might be considered that is voluntary for existing users but includes plumbing code changes that will affect new construction. The next level of conservation might include retrofit measures and rate structure changes; a more aggressive program might also consider turf restrictions. The conservation program should also be targeted to specific customer classes to increase cost effectiveness. Examples might be focusing turf restrictions and/or irrigation planning assistance to multi-family complexes and commercial/industrial users, or public education programs to residential users.

Conservation savings for each program should then be estimated using one of the methods described earlier. Care should be taken not to over-estimate savings as savings by measure are not necessarily additive. The effectiveness of one measure may limit the effectiveness of another measure in the program.

COMPARING CONSERVATION TO OTHER WATER SUPPLY ALTERNATIVES

From an economic standpoint, to maximize the effectiveness of public expenditures, conservation should be assessed on the same economic and institutional basis as traditional water supply projects. This requires treating conservation as a water resource, such as reservoirs, wells, etc. Standard economic methods such as benefit-cost or net-present-value analyses should be applied to each water supply alternative, including water conservation. This will identify the most cost-effective level of conservation.

SELECTING THE CONSERVATION PROGRAM

The economic analysis will determine the most cost-effective level of conservation. However, other factors will affect the conservation program chosen. These include the timing of the water demand, the political acceptability of the most cost-effective conservation, and the impact conservation may have on utility revenues. Conservation affects water revenues by adding the cost of implementing the program to revenue requirements while at the same time reducing the volume of the water sold.

IMPLEMENTING THE PROGRAM

Several entities have bogged down in their conservation programs as they try to get a better handle on the savings that can be achieved from each measure. Research to date has not been effective in isolating savings attributed to individual measures. However, there are many case studies that have demonstrated cutbacks in demand as they implement their conservation program. As a community, it is important to begin the conservation process, even if it creates no impact on existing demand but begins to modify future consumption by new users.

Status of Conservation in North America

Most of the more aggressive conservation programs are currently being practiced in the southwest United States. This area developed conservation practices earlier because of limited water supplies and arid climates. As other areas in North America begin to have water shortages, they are looking to the southwest for practical measures that are appropriate for them.

Table 2 summarizes a recent survey we performed on selected cities in North America. We found very little conservation practiced outside the southwest. Several midwestern cities have adequate summer rainfall that would explain the lack of landscaping measures. The lack of indoor conservation measures may indicate that water supplies are not critically short in these cities.

Survey Results in Canada

In Canada, the southern Ontario cities have focused their conservation programs on educational and public relations measures, and summer watering restrictions during drought conditions. The Waterloo and Kitchener areas have additional landscape-related programs. Toronto and Scarborough have not implemented any programs to date.

In Saskatchewan, Saskatoon has no water conservation plans in place. Regina has an educational and public relations program, and some lawn-watering restrictions.

Calgary and Edmonton both have implemented sonic leak detection, and educational and public relations measures. Edmonton also has provided incentives for low-flow plumbing fixtures. Edmonton is currently contacting many U.S. and Canadian cities regarding conservation programs.

None of the British Columbia Cities has conservation programs in place. However, the Vancouver area water purveyors are assessing the need for conservation, and Victoria is considering promoting conservation through a change in its water rate structure.

Except in Toronto, residential users in southern Ontario are nearly 100 percent metered. In western provinces, significant portions of the Calgary and Vancouver areas are not metered. None of the Canadian cities uses an increasing, or inverted, block rate structure.

Table 2
Summary of Water Conservation Programs in Selected Canadian and U.S. Cities

City	Landscape-related programs	Incentives for low-flow plumbing	Educational and public relations programs	Watering restrictions and ordinances	Leak detection and infrastructure programs	Pricing incentives
Waterloo area	X		X	X		
Kitchener	X		X	X		
Windsor			X	X		
London			X	X		
R.M. of Durham				X		
Etobicoke						
Toronto			X	X		
Scarborough						
Mississauga						
Calgary			X			
Edmonton		X	X		X	
Regina			X		X	
Saskatoon			X	X		
Vancouver area						
Victoria						
Milwaukee						
Minneapolis						
Omaha				X		
Denver	X	X	X			X
Salt Lake City			X	X	X	X
Boise	X		X	X		
Tucson	X	X	X	X	X	X

Survey Results from U.S. Cities

None of the Midwestern cities has an active water conservation program. Omaha will issue free inhouse water conservation kits at the customer's request, and has a publicity program to reduce summer peak usage. Minneapolis banned summer lawn watering during 1988 and 1989 but, due to abundant rainfall, will not ban watering in 1990. Boise and Salt Lake City are gearing up conservation programs but do not yet have specific conservation targets or a method for evaluation programs.

Of the U.S. cities surveyed, Denver and Tucson have the most extensive programs. With the exception of outdoor watering restrictions, however, none of Denver's numerous conservation programs are mandatory, although there are legislative initiatives for building and plumbing codes. Tucson's programs are somewhat more aggressive, including building and plumbing codes, a more severe rate structure, and enforcement of the water waste ordinance.

Milwaukee is the only city surveyed that continues to use a declining block rate structure for the residential sector. Denver, Tucson, and Omaha use an inverted, or increasing, block structure for residential, and the remaining surveyed cities use a flat rate per hundred cubic feet (ccf). Non-residential users in Denver and Omaha are under declining block rates. Tucson has the highest water commodity rate (per ccf). Because nearly all cities have some form of meter charge, Tucson doe not necessarily have the highest water bills, however. Omaha and Tucson both use seasonal pricing.

With the exception of a portion of Denver's residential sector, all users in the surveyed cities are metered. Denver anticipates that all users will be metered by 1993.

CONCLUSIONS

From an engineering perspective, water conservation is perhaps the most flexible resource available because it has very little environmental impact, it can be implemented almost immediately, and it can be targeted to meet the needs of a given demand. Because of the many roles it can play, conservation is also a versatile resource. If water supply is short, conservation needs to be compared to other alternative supplies. It is likely conservation will play a significant role in alleviating water supply problems.

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VALVE ACTUATOR SIZING AND SELECTION

Roy Raftis, Division Manager,
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INTRODUCTION

While this Topic of Valve Actuator Sizing and Selection appears to be very simple on the surface, there are many aspects that must be explored when properly selecting an Actuator for a Valve. Usually the manufacturer will select an Actuator suitable for the Valve, but, quite often if the manufacturer does not have all the information regarding the service, an improper Actuator selection may be made.

What I would like to do is explore some of these requirements so that you can have a better understanding as to all the information that is required to properly size and select a Valve Actuator. Because of the limited amount of time, my talk will be focused on Butterfly Valves. Other valves that exhibit similar characteristics are Plug and Ball Valves. I will also be reviewing the different types of Actuators and their features and benefits.

Butterfly Valves have three types of torque involved:

- packing and bearing torque
- seating and unseating torque
- dynamic torque

PACKING AND BEARING TORQUE

(Slide #1)

Packing and bearing torque are found in all valves. The packing torque is caused by the packing that prevents the media from leaking up or down the shaft bore area. The bearing torque is caused by the pressure or force of the media forcing the internals toward the downstream side.

SEATING AND UNSEATING TORQUE

(Slide #2)

Seating and unseating torque is found in all valves. This is the amount of torque that is required to force the disc into and out of the seat of the valve. The seating and unseating torque is directly proportional to the amount of pressure the valve is trying to shut off against. (Slide 3) in some cases where low pressures are required, the disc of the valve may be undercut so the valve will have a lower pressure rating, but will also require less torque; thus the valve will require a smaller actuator.

DYNAMIC TORQUE

The dynamic torque is caused by the media flowing by the disc on the valve. On a butterfly valve the dynamic torque profile would look like the following, (overhead #1). This is true only if a constant pressure drop is maintained across the valve.

[Overhead #1A]

As you can see, the type of disc and direction of flow also make a difference to the amount of Dynamic Torque. A quick rule of thumb is the seating and unseating torque will be higher in most cases up to 20" valve sizes thus; it is fairly safe in sizing the valves based upon the seating and unseating torque.

An example of high Dynamic Torque is as follows: [Overhead #2] If you have a constant head reservoir and a gravity feed line into a valve. This system will give you a constant pressure drop at all valve openings [less frictional loses in the pipe line].

These types of applications would constitute about 1 or 2% of all applications. [Overhead #3] Most valves that are installed are used in conjunction with a pump type of system.

In this type of application, the pressure decreases as the valve opening increases. As the flow increases the amount of pressure drop required decreases. If we took the reservoir and the pump type of system and plotted them in the symmetrical disc dynamic torque graph, it would look as follows:

[Overhead #4]

As you can see the reservoir type of system will create more dynamic torque than the pump type of system.

TORQUE REQUIRED TO OPERATE VALVE (IN LBS)

[Overhead #5]

Valve Size [inches]	Seating and Unseating Torque in LBS.	Valve Torque at 80 with 5 psi diff Press LBS.	Valve Torque at 80 with 10 psi diff. Press LBS.
4	380	95	190
20	11,000	12,000	24,000
48	160,000	165,000	330,000

On a 72" butterfly valve with the valve in the closed position and with 150 psi pressure on the disc, the force exerted in 610,416 lbs. The weight of a 747 airplane is 800,000 lbs.

THREE MOST COMMON TYPES OF ACTUATORS

The three most common types of actuators are: Pneumatic, Electric and Hydraulic

PNEUMATIC

Uses compressed air to actuate a piston which is converted to a rotary motion. Most air actuators are sized based on a 80 psi supply pressure. The manufacturer has tables or can calculate the amount of torque the actuator will produce at 80 psi and also a higher and lower air pressures.

This type of actuator can be fitted with a spring return feature in order to fail the valve to the open or closed position depending on your requirement. It should be noted that sizing allowances must be made in order to accommodate the spring return feature.

[Overhead #6]

ADVANTAGES

- valve can fail to either direction upon power loss
- infinite modulation
- air is clean and relatively low cost
- usually less expensive
- simplest form of actuator
- ease of maintenence
- generally trouble free
- ease of field repair availability

DISADVANTAGES

- can be big and bulky
- air is compressible
- requires a compressed dry air supply
- upon loss of air, difficult to manually override

ELECTRIC

Uses electricity to power a motor which actuates the valve stem or a gear box on larger valves.

A small electric motor is powered and through a series of gears produces the amount of torque required to operate the valve. The electric actuator incorporates limit switches to stop the motor at the fully open and fully closed position.

this type of actuator also incorporates over torque switches to stop the motor in the event something becomes trapped in the valve.

(Overhead #7)

ADVANTAGES

- can be used where no air is available
- has torque sensing capabilities
- has intregal manual override

DISADVANTAGES

- upon power loss, valve stays in last position
- complex piece of equipment. Some cases not easily field repairable.

HYDRAULIC

Uses a cylinder actuator and is powered by fluid. This is very similar to pneumatic actuators. The fluid can be water or oil and is most times taken from the media that is in the pipeline.

(Overhead #8)

ADVANTAGES

- Valve fail to either direction upon power loss
- capable of very high torque
- liquid is not compressible

DISADVANTAGES

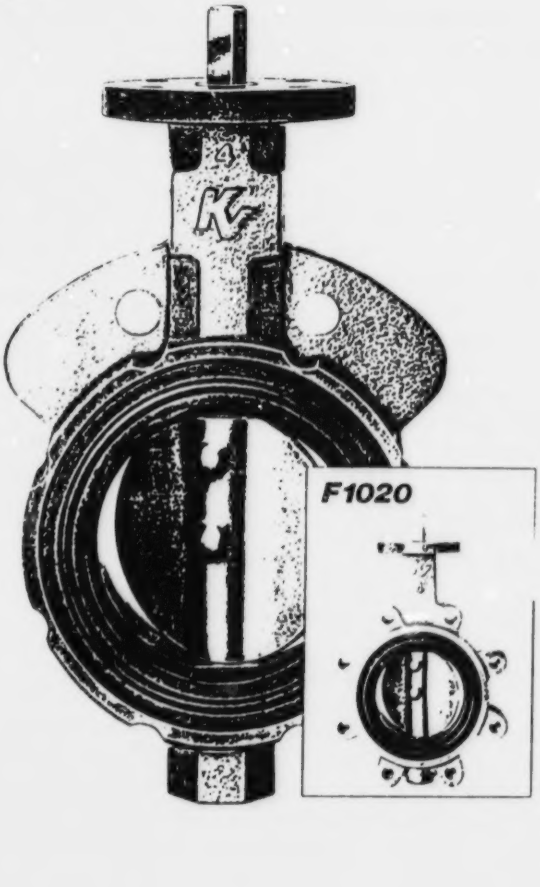
- can be costly to maintain
- fluctuations in supply pressure affect torque
- can be messy if cylinder leaks
- difficult to manually override

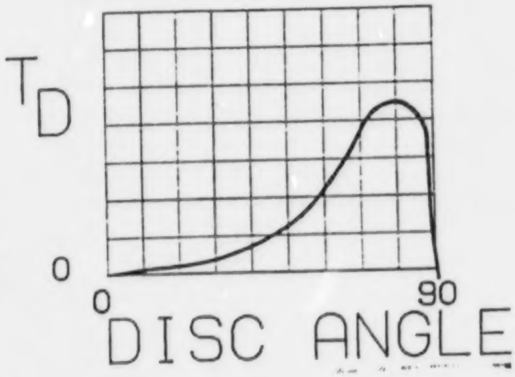
CONCLUSION

In conclusion the owner and supplier must review the application and service that the valve and actuator will be subject to. Together they must pick an actuator and valve combination that will perform for a long period of time.

I would recommend that a 25% safety factor be applied to the maximum amount of torque required for the valve, and pick an actuator closest to this number. Remember to determine which is your maximum torque seating, unseating or dynamic torque.

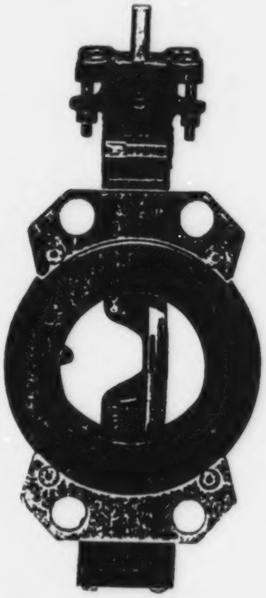
OVERHEAD #1





OVERHEAD #1A

GENERAL PURPOSE K-LOK



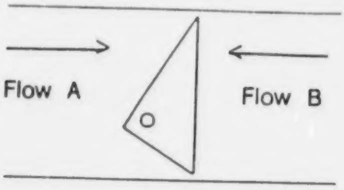

Flow A
Flow B

Fig.1

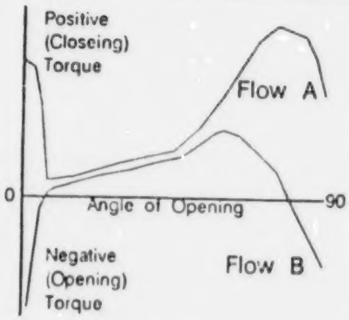
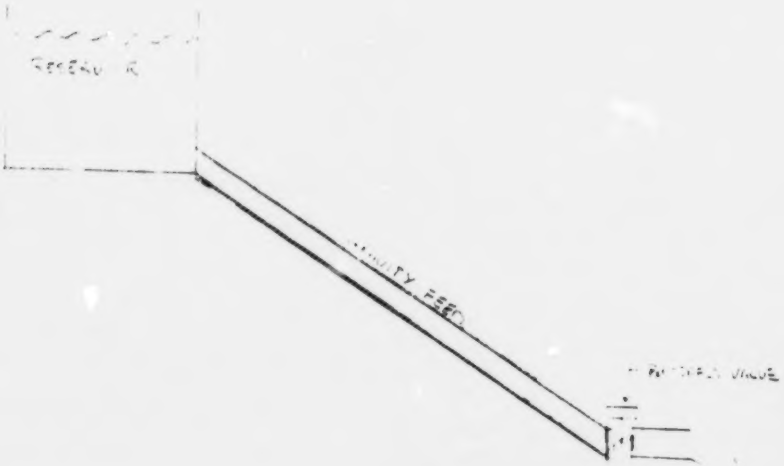
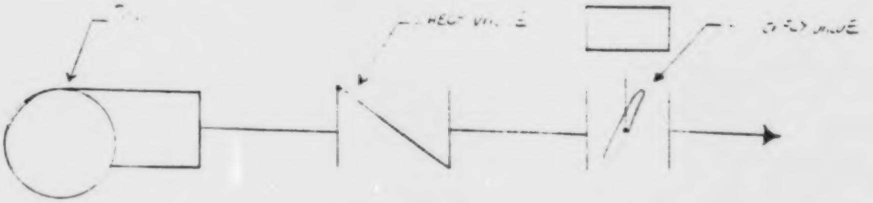


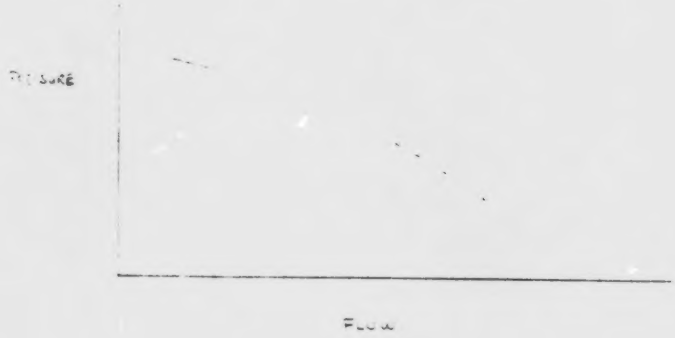
Fig.2

OVERHEAD #2

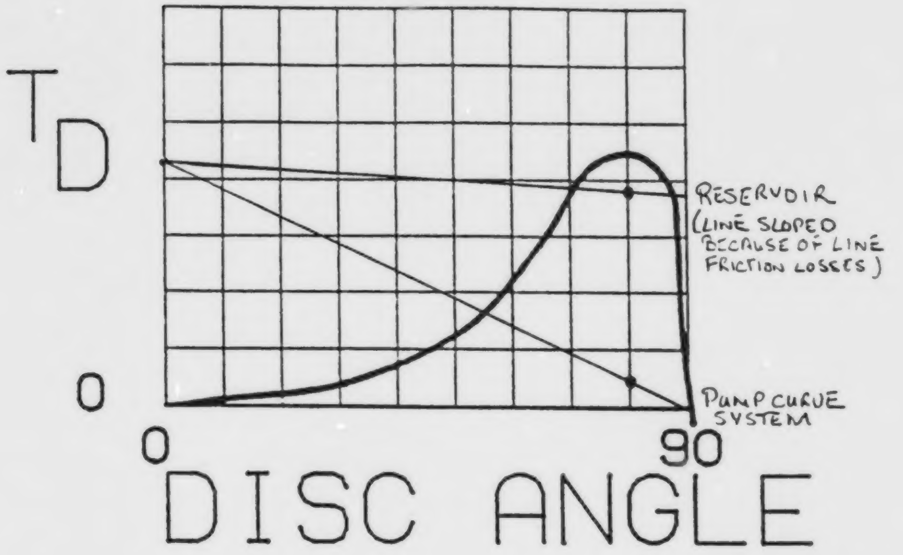

RESERVOIR
GRAVITY FEED
BUTTERFLY VALVE

OVERHEAD #3


CHECK VALVE
BUTTERFLY VALVE



OVERHEAD #4



BUFFALO POUND EXPANSION PROJECT

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Associated Engineering (Sask) Ltd.

Derrick Bellows, P.Eng.
City of Regina

INTRODUCTION

The Buffalo Pound Water Treatment Plant is the major municipal water treatment facility in southern Saskatchewan. It was built in the early 1950s by the cities of Regina and Moose Jaw through the Buffalo Pound Water Administration Board. The plant is impressively located on the bald prairie three kilometres south of Buffalo Pound Lake. Raw feed water originates in the South Saskatchewan River at Lake Diefenbaker. Part of the controlled discharge into the Qu'Appelle River system is destined for the plant.

In its thirty five year history, the plant has gone through numerous changes to increase capacity and improve quality. By the early 1980s, the original plant had been optimized and fine-tuned to its limits. It was determined that the best way to meet water requirements into the next century was to build on the existing facilities. Staged capacity increases were planned for long term growth; and extensive process development and pilot testing was done to address specific water quality problems and improve quality control and efficiency of the operation.

An engineering commission for the initial expansion was awarded to Associated Engineering in 1986. Detailed design was carried out in 1987 and construction began in the spring of 1988. The expanded plant was producing water on July 1, 1989, thus fulfilling commitments to the owner and the public.

What follows is a brief description of the management of this large and complex project. Fittingly, this is a joint presentation. You will be shown two perspectives on management. The owner's perspective will present a "global" picture where management had to identify and control ALL aspects of the project. That included the political and operational, as well as the technical ones. The engineer's perspective will focus on more specific elements of management which could be seen as "technical" or more "engineering oriented" functions.

OWNER'S PERSPECTIVE

Agreement Between Regina and Moose Jaw

For administrative and legal reasons, an "owner" of the project had to be defined. A formal agreement was developed and executed in early 1988. The agreement achieved a number of important goals:

1. It defined a schedule and a result for the proposed work. Simply put, a Phase I expansion to the plant would be operational by July 1, 1989, and would change plant output from 135 (million litres per day) Ml/d to 177 Ml/d, a 30% increase.

2. It defined the City of Regina as the "owner" having the rights and responsibilities for all project management and administration required to achieve the defined objectives.

3. It detailed commitments made with Associated Engineering for services provided in predesign, detailed design, and general and resident engineering during construction. This, along with the Engineering Agreement, provided the basis for the working relationship between the City and Associated Engineering.

4. It established a cost sharing agreement between Regina and Moose Jaw and used this and project estimates to predict expenditures by contract and year. As well, miscellaneous costs were also identified.

5. It defined water allocations to Regina and Moose Jaw for the increased plant capacity.

6. It laid out specific requirements for project financing and accounting. As shown later, this became a significant part of the owner's management duties.

The agreement became the corner stone around which the management and administration of the project was built.

Structures and Relationships

With the agreement between the cities in place, it was necessary for the City of Regina to address how best to meet its obligations for management and administration of the project. This mainly involved designating sufficient staff time, integrating the project into existing corporate structures and creating new structures and relationships for the peculiar requirements of this project.

The Water Division of the Municipal Engineering Department was charged with accomplishing these goals. First, the position of Waterworks Engineer was dedicated to the major project work. This primarily consisted of the treatment plant expansion and the separate but related work on twinning of the water pipeline between the plant and Regina. For the two year duration of the project, the normal duties of the Waterworks Engineer were carried out by the Management of Water Engineering and an engineer hired on a term basis.

Existing corporate structures were compatible with the requirements of the project. Normal procedures and policies regarding reporting, approvals, tendering, awarding, purchasing, and payments formed the basis for project management for the City.

Some aspects peculiar to the project did require special considerations.

- Paper flow was streamlined as much as possible without jeopardising the maintenance of proper record keeping. This was particularly true of repetitive functions such as Change Price Requests and Change Orders.

- Special arrangements were made with contractors to ensure timely payment of regular progress claims. This was especially important during the peak of construction when individual claims exceeded $1,000,000 per month.

- Special arrangements were made for project financing with the creation of a separate bank account. Total project cost projections were done quarterly to identify financial needs. On the basis of these forecasts, the two cities provided funding to ensure a positive bank balance. A review of expenditures was done quarterly to control cash flow through the account and as a check on the accuracy of the forecasting.

HIGHLIGHTS OF THE PROJECT

A few specific examples will follow to illustrate the breadth of management required for this project.

Co-ordination with Operations

Like any project which involves an existing operation, the consequences of construction activities which could effect the operations were of paramount importance. Throughout the course of construction, numerous shutdowns and operating disruptions were required for tie-ins and equipment replacement on pipelines, structures, utilities, and plant services. In all cases, exhaustive planning was done with contractors and operations to ensure that the work required was accomplished at the best time, in the most efficient manner, and with fall back positions that would guarantee the reinstatement of service if the work could not be completed. In some cases, the planning even included reversing the flow in the Regina pipeline to bring water from the Regina well fields back to service Moose Jaw.

Changes Caused by Actual Construction Conditions

Again, like most construction projects, what you find on site is sometimes different from what you expected to find. In our case this is best exemplified by the connection between the new construction and the existing raw water lake intake. What was designed and bid as a "dry" connection was actually done "wet" by divers because the existing line was different from what was expected. The major management challenge was to achieve a suitable redesign, control costs, maintain schedule, and to get the work done safely and effectively. Properly utilizing and co-ordinating the expertise of all participants (contractors, operations, and engineering) was the management function in this case.

Project Modifications

After construction was started, a contractor made the recommendation to significantly change the control system. The changes would benefit the plant by adding automatic control through the new SCADA system to parts of the old plant. Extensive detailed analysis, negotiation, and redesign were carried out to achieve this change and still maintain the original design intent and control philosophy. Again, co-operation and commitment by the engineers, contractors, and plant operators was necessary. Management kept the process focused on the desired goal and balanced the respective benefits and costs for each of the parties involved.

Public Relations

The expansion of a municipal water system located in a dry region, and during a multi-year drought is subject to considerable public interest and scrutiny. Regular reports were made to keep officials and the public abreast of construction progress. Special functions were planned for sod turning, first new water, and a grand opening of the completed project.

In this area, management moved into somewhat unfamiliar territory; dealing with the media and organizing a public event involving guest celebrities, caterers, large tents, busses and 400 school children. Expert assistance was found in the City of Regina, Public Affairs Department and through a specialized consultant.

ENGINEER'S PERSPECTIVE

Expansion Criteria

All components are to be sized for the ultimate expansion to 275 Ml/day except as noted:

- New lake intake (deferred)
- New low lift pump house (pumping 205 Ml/day)
- New substation at low lift pump house
- New supply line to Buffalo Pound Water Treatment Plant (deferred)
- Investigation of existing supply line
- Review of treatment problems associated with oxygen and nitrogen saturation in the water supply, particularly during the winter
- Review of deaeration and aeration alternatives
- Review of rating of existing clarifiers
- New clarification equipment (205 Ml/day)
- New filtration equipment (205 Ml/day)
- New carbon contactors (205 Ml/day)
- Improvements to:
  - chemical feed, mixing and application systems
  - laboratory
  - operating area
  - control room
- Completion of plant waste water management system (205 Ml/day)
- Maintain existing operations during construction
- Increased water was to be available by the summer of 1989

The expanded plant will include cascade aeration for reduction of supersaturated oxygen conditions during the winter and for some reduction in volatile organics.

Tapered flocculation is followed by high rate up flow clarification and rapid sand filtration.

Taste and odour control and trihalomethane reduction is achieved by the use of granular activated carbon (GAC) contactors, supplemented at times with powdered activated carbon. The GAC is regenerated on site by a fluidized bed furnace.

The finished water pH is adjusted upwards by the use of soda ash and an air stripping system. The air stripper also assists in reduction of nitrogen trichloride.

Associated Engineering was commissioned to provide engineering services for predesign, design, and construction.

Key personnel in all positions were backed up to maintain continuity throughout the project, including:

- City of Regina and Buffalo Pound Water Treatment Plant staff
- Associated Engineering Design Team
  - Project Manager
  - Process Engineer
  - Electrical and Controls Engineer
  - Project Engineer
- Contractors representatives

All of the above were backed up and supported by the Associated Engineering Design Review Board.

The intent was to do as much work as possible locally to ensure future support and technology availability. In fact, over 98% of all engineering was done locally.
To assist in the management of the project, a Project Procedures Manual was prepared. The Project Procedures Manual was based on the City of Regina's Terms of Reference, discussions with the City of Regina and Buffalo Pound Water Treatment Plant staff, and Associated Engineering's drafting standards and project procedures.

This manual defined the contacts, criteria and quality requirements for the project.

In addition to the project procedures manual, computer aided drafting (CAD) and databases were used to increase efficiency. CAD allowed the quick revision of concept details and provided the opportunity to evaluate more configurations and update changes quickly. Computerized databases were invaluable in documentation, review, and updating of activities and costs in a timely manner. The resultant was that project personnel had more time to manage the project.

The essence of project control is to maintain timely action by management to keep the project on its planned track.

Key elements of the project control include:

Scheduling

Activity oriented scheduling methods were used (functional charts identifying key activities and dates and showing actual progress against planned activities).

Cost Control

Cost control made use of all sources including:

- Cost information available from other recent projects
- Published Cost data
- Manufacturer's quotes

Ongoing cash flow projections were provided during construction including all contract changes.

These cost control methods have proven effective by maintaining the total contract cost within the budget.

Project Accounting

The engineering project accounting system provided: man hour expenditures with support data, disbursement allowances, and comparison between actual and projected costs.

Construction Management

Construction management was used to:

- Facilitate administration of contracts
- Provide full documentation of contract changes
- Monitor and record daily construction activities and ensure quality
- Document commissioning activities
- Record field changes
- Recommend process payments against contract accounts

Activities to ensure effective communications and management were carried out throughout the various stages of the project.

Predesign

- Taped meetings were held with operating personnel prior to concept development
- The Criteria developed from the Terms of Reference, preliminary studies, and meetings with the operating personnel, Technical Steering Committee and Design Review Board was reviewed
- Concepts were developed
- Concept development was reviewed with the owners and operating staff
- Required testing by the plant and outside organization was defined
- Required evaluations by operating staff and consultants was itemized
- A design basis report was prepared including an estimate of costs
- Total costs were reviewed to meet the owner's budget and a report on rescheduling of capital costs was prepared to identify construction which could be deferred

Design

- The design was reviewed with the City staff, plant staff and the Design Review Board
- A detailed review by the City and plant staff was conducted at the key design stages of 50% and 90% completion
- Input was obtained from suppliers
- An operating philosophy was developed
- Regular meetings were held with the City and operating staff
- Cost estimates were prepared prior to tender

Construction

- Shop drawing review was provided to ensure conformance with the design intent
- Meetings were held with the suppliers' representatives to review the design intent and contract requirements
- contractors' and suppliers' proposals were evaluated based on their merit. Contractors' proposals resulted in a significant increase to the control component while still maintaining the project within budget
- Resident inspectors were maintained on site during key construction periods to assist in interpretation of documents and assure quality
- Construction photographs were taken throughout the project to document progress and details of construction. [At the project completion a set will be turned over to the owner]
- Construction meetings were held weekly during critical construction periods
- Monthly meetings were held with the owner to review the status of the project

Post Construction

An Operating Manual was prepared with input and review by plant staff. The manual was based on the operating philosophy developed during design and the manufacturers' maintenance manuals.

Contract Record Drawings were prepared indicating changes made during construction. They were based on field records and the contractor's submission of record drawings.

These activities were carried out throughout the project. However, they do not stop here.

Plans are being discussed to have post construction technical and management reviews. The technical criteria will be evaluated to determine: what worked, what wasn't effective, what can be improved, what can be done now, what must be deferred, and to identify items discovered during construction which were not included in the design. The management system will be reviewed to determine what worked well and what could be improved.

The follow-up review should include a review of operating procedures, possible improvement, concepts that worked well, and concepts that required improvements.

The management of the project was key to bringing the project in on budget and on time. Water was available from the new works and turned on manually the first Monday in July, 1989. This was accomplished even though several components were expanded beyond the original scope including: controls, one additional pump, and one additional travelling water screen at the raw water pumping station and one additional clarifier. The total value of changes was limited to 2.25% of the tendered quotations which included 0.32% for additional work and 0.67% for one change due to holes in the existing buried intake line at the raw water pump house.

The management of a project is necessary to the successful completion of a project. A well designed project must have the proper management throughout to make the project successful. The involvement of all parties is an absolute necessity. The best designed plant will not function properly unless the operating staff understand the system and are happy with the design. We are very fortunate to have staff from the City and the plant who exhibited a professional and personal interest in this project. The operating staff were involved from a meeting held prior to predesign, right through construction and optimization.

Every project is different, no management system is perfect and the system must be continually evaluated and updated to meet the latest standards.

CONCLUSION

It has been said that project managers share one over-riding concern - risk. Successful project management overcomes risk with superior planning, team building, and strong controls. These components were all evident in expansion to the Buffalo Pound Water Treatment Plant.

Planning was farsighted and comprehensive. It involved experts from the cities of Regina and Moose Jaw as well as plant personnel and consulting engineers.

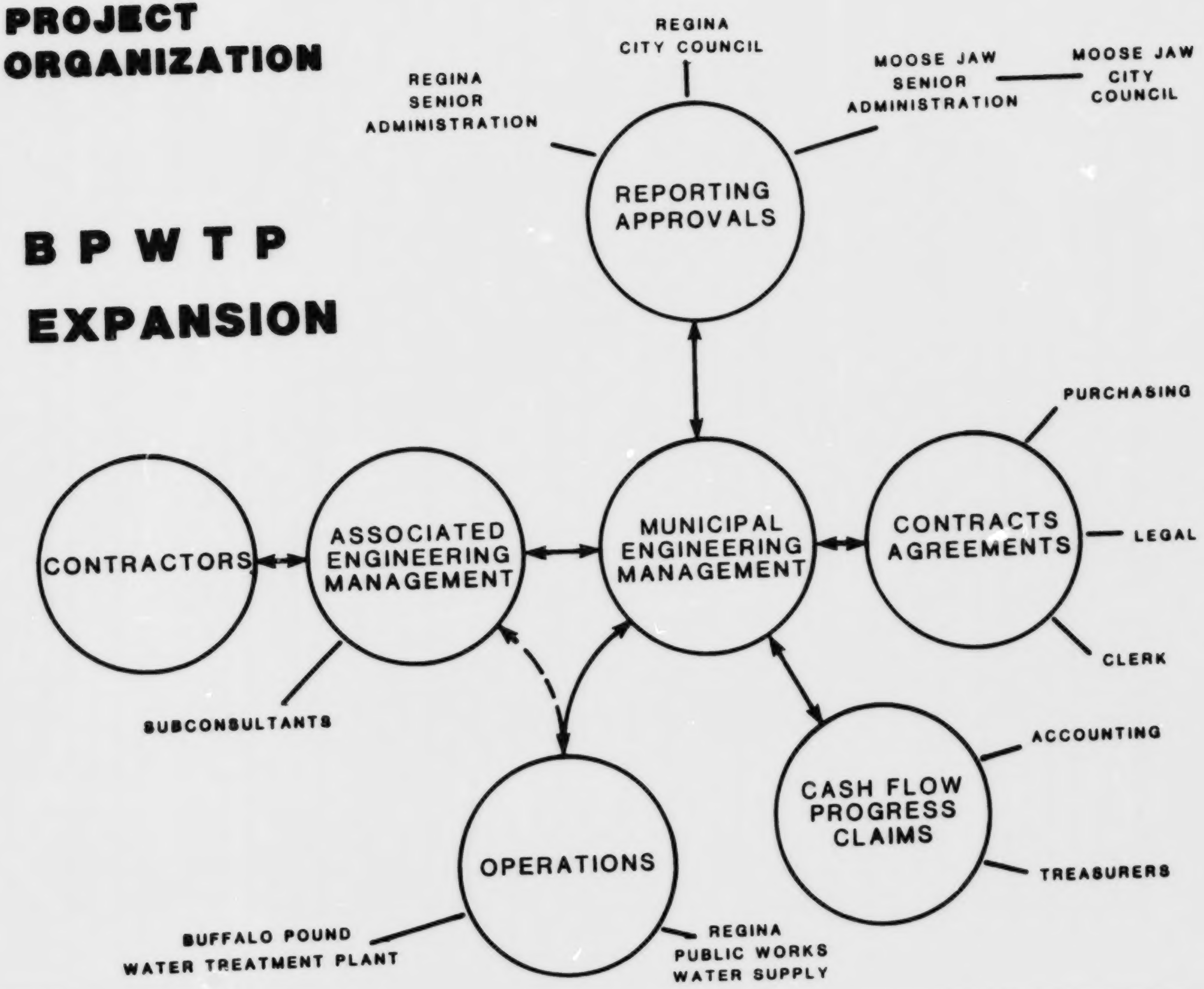
Team building was most evident during the actual construction phase. The expertise and credibility of contractors, engineers, operators, and managers was quickly established and this allowed each to do their job effectively and efficiently. There was very little unwarranted overlap and very few "cracks" for things to fall between.

The structures and controls described above were the primary management tools used to monitor progress and quality of the work, and to respond to changes and unknowns discovered during the work.

Of course, the success of virtually all projects is measured by the simple parameters of cost and time. A public commitment by our politicians for water availability defined the critical schedule dates; and that commitment was met. The initial project budget established in 1986 was not exceeded. We are proud of meeting those goals. However, we are also proud that numerous, significant modifications were made to the project which resulted in a better finished product. A devoted team and effective management system were crucial in meeting these goals. This water treatment plant is and will remain a visible testimonial to those individuals and organizations who were committed to build the best facility that they could.

PROJECT ORGANIZATION

B P W T P EXPANSION


REGINA CITY COUNCIL
REGINA SENIOR ADMINISTRATION
MOOSE JAW SENIOR ADMINISTRATION
MOOSE JAW CITY COUNCIL
REPORTING APPROVALS
PURCHASING
CONTRACTS AGREEMENTS
LEGAL
CLERK
CONTRACTORS
ASSOCIATED ENGINEERING MANAGEMENT
MUNICIPAL ENGINEERING MANAGEMENT
SUBCONSULTANTS
ACCOUNTING
CASH FLOW PROGRESS CLAIMS
TREASURERS
OPERATIONS
BUFFALO POUND WATER TREATMENT PLANT
REGINA PUBLIC WORKS WATER SUPPLY

DESIGN CONSIDERATIONS FOR EXPANDING THE CITY OF REGINA SEWAGE TREATMENT PLANT FACILITIES

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Manager, Environmental Engineering Division
Municipal Engineering Department
City of Regina

INTRODUCTION

The City of Regina's present sewage treatment plant is nearing capacity and requires expansion. In addition, the present process is projected to be unable to meet future discharge criteria and thus also requires process upgrading. Present nominal capacity is between 85 and 90 Ml/d. The present process consists of conventional primary treatment followed by aeration lagoon secondary and tertiary treatment for phosphorous removal using alum. Treated effluent is discharged into Wascana Creek which flows into the Qu'Appelle River system.

PLANNED EXPANSION AND UPGRADING

Based on a conceptual study done in 1987, the City has adopted in principle an upgrading and expansion plan that will provide for an expanded primary plant, a new biological nutrient removal secondary plant, associated sludge handling and disposal and disinfection systems. The expansion and upgrading to the full biological nutrient removal may require a two phase approach depending on the findings of the upcoming functional and design basis engineering studies and the projected capital costs. If necessary, the secondary would be designed as a conventional activated sludge process with conversion to full biological nutrient removal at a later date. Should this be the case, alum addition to the secondary clarification process would be required for phosphorous removal until the full biological nutrient removal conversion.

SOME FACTORS TO BE CONSIDERED IN THE PROJECT DESIGN

As with any sewage treatment plant expansion and upgrading there are always many factors which must be considered by the designer, many of which are specific to the location. In Regina, there are a range of factors which will be relevant and a brief review of some of the more important ones are outlined.

1. Climate

The climate in Regina causes the winter temperature of the raw sewage to be quite low. Based on the records available a temperature of 10 degrees C can be expected for upwards of a week during a cold spell. This factor will be important in the design of the biological treatment, in particular the nitrification/denitrification process which are very temperature sensitive. Bench scale or pilot work will be necessary to establish nitrification/denitrification rates that can be expected for Regina's sewage.

In addition, Regina experiences the typical prairie rainstorms that can add major quantities of stormwater to the domestic sewer system over a relatively short period of time. Regina still has some combined sewer systems in the older part of the City and several areas where roof leaders remain connected to the sanitary sewer. The City collection system conveys all of the sewage to the main McCarthy Pump Station which pumps the sewage to the treatment plant about 5.0 kilometers west. The present capacity of the station and forcemain is 150 Ml/d and will be upgraded to 300 Ml/d to minimize overflows to Wascana Creek. The new forcemain is being installed this fall and the pumps will be upgraded in 1992. The new sewage plant will have to be designed to manage both the dry weather and wet weather flows.

2. Flat Sewers and High Sulphate Levels

The relief in the City of Regina is very small resulting in extremely flat sewers. This results in sewage that tends to have elevated sulphide levels particularly in the summer. The sewage forcemain from McCarthy Station to the sewage treatment plant also causes opportunity for additional sulphide formation. The design of the treatment plant will have to provide for means to control, manage and possibly treat the raw sewage for sulphide to avoid costly plant corrosion and occupational health concerns.

One benefit of the flat sewers is that the stale sewage tends to have a considerable amount of volatile fatty acids which has been found to promote the biological nutrient removal process. The levels measured in the sewage arriving at the plant in the few samples to date were as follows:

November 20, 1989	Morning	15.8 mg/l
	Afternoon	4.8 mg/l
January 29, 1990	Morning	10.4 mg/l
	Afternoon	8.7 mg/l

The design will have to take into consideration the volatile acid levels to assess the need for a sludge fermenter to augment levels to provide for good nutrient removals.

The City sewage contains relatively high levels of sulphate due to the groundwater component of the water supply. It may be necessary to assess the importance of this on the biological nutrient removal process by means of bench or pilot trials. High sulphate could lead to odorous conditions in the anaerobic and anoxic sections of the biological nutrient removal process.

3. Receiving Environment Requirements

The future treatment plant will be required to provide an effluent that produces a nontoxic effluent at the confluence of the Wascana Creek and the Qu'Appelle River in terms of unionized ammonia. Also, a phosphorous level in the plant effluent of 1 mg/l or less on an average annual basis will be required. Depending on the results of an ongoing aquatic study of the Qu'Appelle River system in the reach affected by the treated effluent, it may be possible to consider a seasonal variation in the level of nitrogen treatment required. At present it appears that very few if any fish overwinter in the Qu'Appelle River between Wascana Creek and Pasqua Lake.

Initially it was thought that nitrogen removal would be beneficial to the downstream lake systems from a eutrophication standpoint. However, the expert limnologists advise that because of the high phosphorous levels in the lakes, removal of nitrogen would not result in less algae production. In fact, they caution nitrogen removal could change the N:P Ratio in the lakes and type of predominate algae could shift towards the blue green types which would not be desirable. The City intends to have this question examined further in the expansion studies to ensure that biological nutrient removal treatment will not result in any unexpected or undesirable conditions in the downstream lake systems.

4. Grit Removal

The present aerated grit removal system will require expansion. In addition a process change may be necessary since the addition of oxygen to the head end of a biological nutrient removal plant can be detrimental to the anaerobic and or anoxic processes. This factor will require careful planning in the expansion and upgrading engineering.

5. Sludge Handling and Disposal

At present the plant has three primary sludge digesters. The expansion will generate more primary and new secondary sludge. Anaerobic digestion of biological nutrient removal sludge does not appear to be a proven process. Some of the literature suggests the secondary sludge can be digested without significant release of the phosphorous whereas other researchers suggest anaerobic digestion will result in the majority of phosphorous being released. This aspect of sludge handling will require careful assessment.

Primary sludge at present in Regina is dewatered and hauled to the municipal landfill where it is used for cover dressing. The City is at present carrying out demonstration trials of sludge addition to agricultural land on a very small scale. In the event that the secondary sludge from a biological nutrient removal plant could not be anaerobically digested, disposal to land may present a problem. Usually regulatory agencies require digestion for pathogen destruction prior to land disposal. Aerobic digestion is very expensive and has more recently been found to have limitations in pathogen destruction. Also landfilling of sludge usually requires that the sludge be digested. Thus in the case of Regina, the sludge disposal option is not going to be a simple proposition and will require some innovative design and planning.

6. Disinfection Process

At present, the City is required to disinfect the treated effluent during the open water season since Wascana Creek downstream is used quite extensively for irrigation purposes. The City presently uses chlorine for disinfection. Since chlorine has come under close scrutiny in recent times from the standpoint of causing potentially harmful chlorinated organics, it will be important to examine the alternates available for effluent disinfection. Processes such as ultraviolet light and ozonation are being applied elsewhere successfully. It will be necessary to examine whether these alternate processes can offer any advantages in the case of Regina.

7. Ultimate Disposal by Effluent Irrigation

The City maintains an interest in effluent disposal by irrigation on agricultural land because it makes sense in an arid climate like Southern Saskatchewan. The problem for Regina is that the cost for a major scheme, which would use upwards of 10,000 hectares of land, is beyond the City's financial capability to pursue on its own. In addition the schedule to develop such a project that would allow time for full farmer participation could be upwards of a decade or more. Notwithstanding, the future plant will be designed where possible to allow for ultimate effluent disposal to agricultural land.

8. Septage Wastewater

Outside Regina City limits, there are numerous operations on septic holding tank systems. private haulers service these businesses and dispose of the septage to the McCarthy Pump Station. The quantity ranges from a low of 50 m^3/month to 3800 m^3/month averaging 1400 m^3/month. Not all of this liquid is domestic sewage. Some of the liquid hauled are industrial type waste waters from the North Industrial area in the R.M of Sherwood. The present disposal point at McCarthy Station is not ideal in that it is located very close to a residential area and is subject to periodic odour complaints. In the sewage treatment plant expansion, the City intends to consider relocating the discharge point to the headworks of the plant. It will thus be necessary to consider the potential impact of this quantity and type of wastewater on the plant operation. In some Cities, the impact of septage wastes has required special treatment to avoid shock impacts on the secondary process.

9. Commercial/Industrial Wastewater

In Regina, the number of industries generating significant quantities of industrial/commercial type wastewater is not large. Based on fairly recent data, the quantity of industrial wastewater from the larger industries totals about 6.8 Ml/d. The volumes are probably somewhat higher if one considers the quantity of wastewater recorded by the City billing procedure as being of commercial/industrial origin is about 28 Ml/d. The design will have to take into account the industrial and trade wastewater both in terms of present and projected future quantity and quality. Some industries pretreat their wastewater to a fairly high standard such as the refinery/upgrader. Industry is also known however, to have periodic upsets which can impact on the overall municipal sewage quality.

10. Population Projections/Per Capita Sewage Volumes

Since the 1987 study completed, the City's population projection have changed quite dramatically. The City is promoting water conservation measures and with the continued emphasis in North America on "waste reduction", it may be expected that the per capita use could level out or even fall over the next few decades. The most recent data in Regina on per capital rates and population projection is as follows:

	Per Population	Capita l/d	Flow (Ml/d)
1985	173,000	409	71
1986	174,000	411	72
1987	176,000	405	71
1988	178,000	390	69
1989	179,000	420	76
1991	182,000*	450*	78*
1996	194,000*	450*	88*
2001	200,000*	475*	90*
2011	205,000*	475*	99*
2016	213,000*	475*	102*

* 1990 Projection

CONCLUSIONS

There are a number of important factors specific to Regina and to the biological nutrient removal process that will require detail consideration in the design of the plant. These factors will also present a challenge to both the designers and operators. It is expected that by the time the Western Canada and Water and Wastewater Conference is held in Regina, the City will have made substantial progress towards development and construction of a new state of the art advanced wastewater treatment plant.

HIGHLANDS TRUNK COMBINED SEWER UPGRADING

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INTRODUCTION

The City of Edmonton is presently served by a sewage collection system which transmits the majority of sewage flows to the Gold Bar Wastewater Treatment Plant. The Plant is located on the south bank of the North Saskatchewan River downstream of the City's centre core. Most wastewater arrives at the Plant through two major trunk lines; the Capilano Trunk Sewer from the City's south side, and the Highlands Trunk Sewer from the City's north side. In 1985, the City decided to undertake the construction of a new trunk line which would connect the north Highlands Trunk line to the Gold Bar Plant, with a connection also to the south side system.

PROJECT OBJECTIVES

The new Highlands Trunk Sewer would fulfill the following objectives:

1. To provide a second river crossing, so that flows could be diverted and inspection safely carried out on the existing River crossings.

2. To provide a reduction in combined sewer overflows in wet weather, by utilization of all of the Gold Bar Treatment Plant capacity.

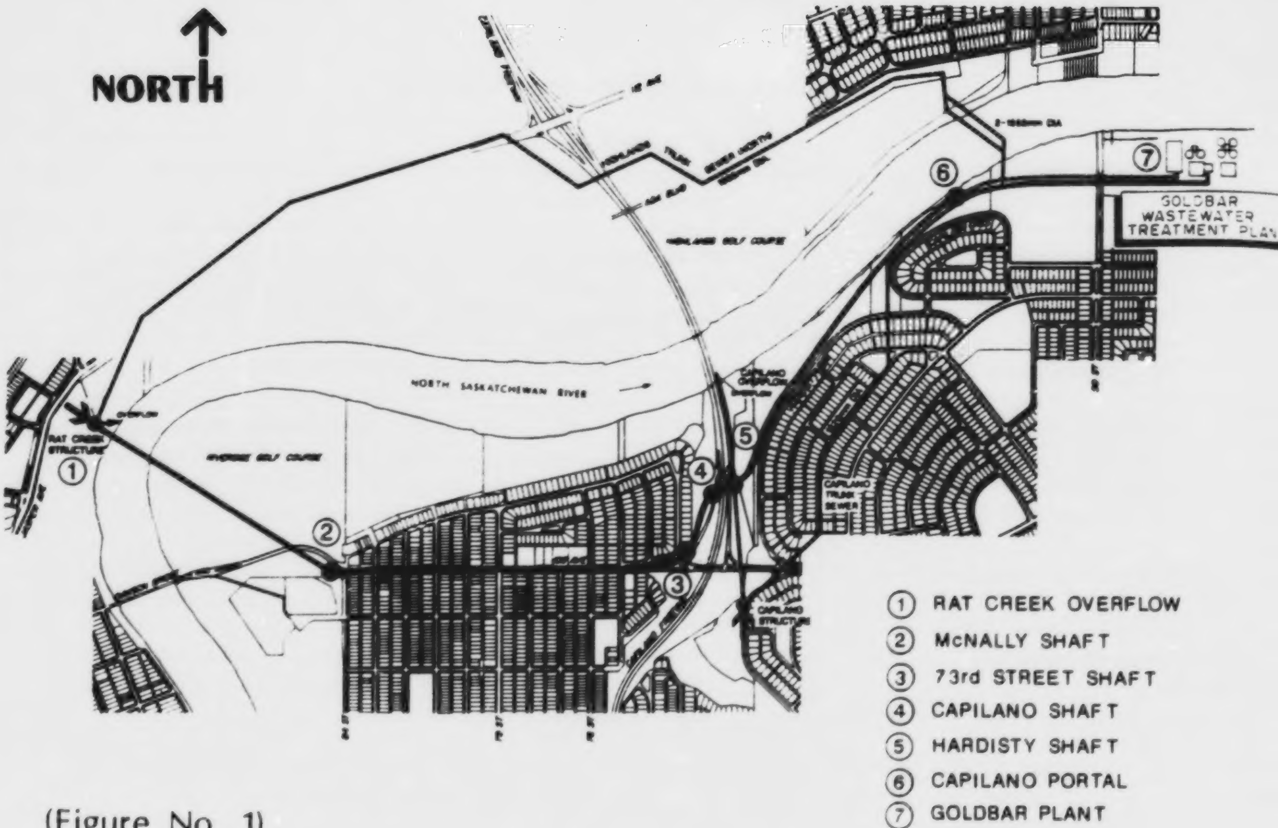
In addition, the new trunk sewer would provide the opportunity to provide additional dry weather flow capacity to the north side and south side systems.

An environmental impact assessment for the project was performed and approved by City Council. The recommended route for the trunk sewer was approximately 4.2 kilometers in length, and was, for the majority of its length, to be tunnelled. Because of the environmentally sensitive nature of Edmonton's river valley, this construction method was seen to be the least damaging and least disruptive to natural habitats, parks and trails located there.

DESIGN CONSIDERATIONS

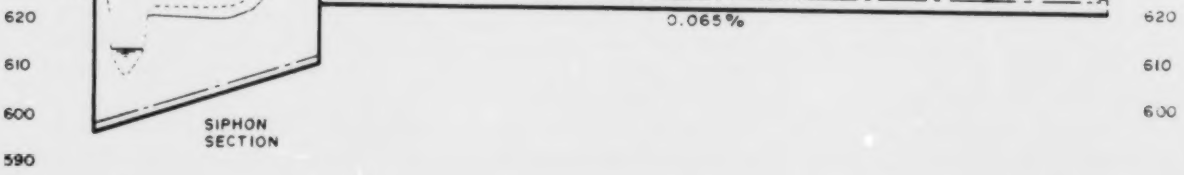
The alignment and grade of the new trunk sewer had constraints imposed on it by the existing system. The downstream elevation at Gold Bar Treatment Plant was fixed, the location and elevation of the sewer's tie-in to the Capilano Trunk's overflow line was limited, and the upstream connection near the existing Rat Creek Outfall was also limited in location and elevation. Within these constraints, the new trunk sewer had to have a minimum inside diameter of 2.25 meters in order to have the required flow capacity, with a grade of .065% to maintain cleansing velocities.

A route was selected so that it avoided existing deep trunk lines, avoided passing under private property, and could be constructed by the conventional tunnelling methods. The depth of cover along the selected route was an average of 25 to 35 meters, with a maximum depth of approximately 42 meters at one location.


NORTH
NORTH SASKATCHEWAN RIVER
RAT CREEK STRUCTURE
OVERFLOW
RIVERSIDE GOLF COURSE
HIGHLANDS GOLF COURSE
CAPILANO OVERFLOW
CAPILANO TRUNK SEWER
CAPILANO STRUCTURE
GOLDBAR WASTEWATER TREATMENT PLAN
① RAT CREEK OVERFLOW
② McNALLY SHAFT
③ 73rd STREET SHAFT
④ CAPILANO SHAFT
⑤ HARDISTY SHAFT
⑥ CAPILANO PORTAL
⑦ GOLDBAR PLANT

(Figure No. 1)

GENERAL SITE PLAN SHOWING EXISTING SYSTEM AND LOCATION OF NEW HIGHLANDS TUNNEL PROJECT.


STAGE IV
960m
STAGE III
1536m
STAGE II
1292m
STAGE I
450m
670
660
650
640
630
620
610
600
590
CLAY
TILL
SHALE BEDROCK
SAND
0.065%
SIPHON SECTION
①
②
③
④
⑤
⑥
⑦

(Figure No. 2)

GENERAL SOIL STRATIGRAPHY AND TUNNEL PROFILE

The geology of the proposed tunnel route was, for the most part, suitable for construction by the conventional tunnelling methods. It consisted of a carbonaceous clay shale, with deposits of coal, an occasional sandstone layer, and layers of bentonitic clays. The exception to this occurred in the area of the tie in with the Capilano Trunk overflow line. Here the trunk line crossed the Capilano Freeway, which is built in a small ravine. Boreholes showed the present of sand, and concerns were raised over possible ground loss if the tunnelling operation, which could cause settlement of roadway above. For this reason it was specified that the 70 meter long portion under the freeway would be hand excavated. It was felt that this method could provide better ground loss control for running sand.

CONSTRUCTION

The City of Edmonton's Public Works Department provided estimated costs to complete Stages 2 and 3 on a unit price and lump sum basis. Their expertise and costs were evaluated against existing construction industry standards, and it was recommended they perform the work.

The Public Works Department's Construction Branch has a history of tunnelling experience, which has developed over the past thirty years. From it's beginnings, when tunnels were largely constructed by hand excavation, it developed its own mechanical tunnel boring machines, purchased some crude open wheel machines, and now owns four shielded TBM's of sizes from 2.5 meters in diameter to 6.2 meters in diameter. The TBM's are manufactured by LOVAT Manufacturing in Toronto.

The construction of Stage 2 of this project began at a tunnel portal in a park just west of 50th Street, in July of 1988. The normal method of lining the excavated tunnel used by Construction Branch in the past, was to first erect steel ribs and wooden lagging as a temporary liner, then to cast in place a concrete secondary liner on the second pass through. The liner used in this project was a precast concrete liner. It consists of four arcs or segments which form a circular ring, one meter in length. These segments are assembled in the trailing shield of the TBM, and expanded to contact the earth after the TBM thrusts ahead. Sealant is placed between all joints, and the expansion joint and circumferential joints are also grouted.

The segments fit together by means of knuckle joints. This allows the ring to deform somewhat with any non-uniform confining earth pressures, and reach an equilibrium with the soil structure around it, so that long term loads are primarily compressive. The thickness of the lining required is only 110 mm.

The use of this precast concrete liner had at least three distinct advantages. It resulted in a large internal pipe diameter compared to the excavated diameter; it allowed for one pass to complete the final lining, which made the process faster; and it was the least expensive system.

The contractor set the alignment and grade for the tunnel by normal survey means, and it was maintained by means of a laser. The mole operator simply watched the beam on a target at the front of the TBM for both line and grade control. The length of Stage 2 tunnel was approximately 1300 meters, which included 3 curves, and the tunnel was kept within the specified tolerances of ± 100 mm for line, and ± 40 mm for grade up to the point of TBM recovery. Similar accuracy was attained in Stage 3.

The tunnelling operation was not without it's problems. On one occasion the TBM hit a small, unmarked, and abandoned coal mine. It was only about fifteen meters in length, but the voids had to be backfilled with a sand/cement mixture to ensure the proper structural behaviour of the precast liner, and it caused a slowdown in production. In another stretch of approximately 100 meters, the clay shale in which the TBM was tunnelling had cracks and fissures which contained waterbearing sands and gravel. When this material was excavated it could not be controlled, and resulted in voids being created above the TBM. They too had to be backfilled with a sand/cement mixture before the segmental liner could be installed. Fortunately, the tunnel was passing through a park area, where there was easy access for a drill rig to find these voids.

Over a period of 20 months, Construction Branch installed approximately 2700 meters of 2.3 meter diameter tunnel at rates averaging about 11 meters per day. Total costs were just under $8 million, for Stages 2 and 3, of which tunnelling costs were about $2500 per meter.

MONITORING PROGRAM

Part of the consultant's work was the institution of a settlement monitoring program. The purpose of the program was to measure ground settlements due to the tunnelling, and compare them to predicted settlements or allowable settlements. This would provide a basis on which to alter excavation and lining procedures if necessary, and also provide data for the owner to counter any damage claims which may arise.

A total of five locations were chosen, and settlement measurements were made using multipoint magnetic extensometers. These instruments produced readings of settlements at various depths above the tunnel, and when placed perpendicular to the tunnel alignment, also gave a profile of the settlement trough created by the tunnel.

Typical settlements measured just above the crown of the tunnel were approximately 20 mm. Surface settlements were in the range of 10 to 50 mm. This was within expected tolerances, and confirmed proper performance of the structure, and proper installation techniques. No damage claims were received on this project.

In addition to the settlement monitoring stations, there was also a station where actual compressive tunnel loads were measured in four segmental rings. This was done with load cells placed between segments of the rings. At several locations, convergence measurements of the diameter of the tunnel were taken. Highly accurate tape extensometers measured the change in horizontal and vertical diameter of selected rings. The results of the load cell measurements and convergence measurements helped confirm the design criteria used for tunnel loads.

SUMMARY

By utilizing an experienced contractor, and an efficient and appropriate design, the City of Edmonton was able to achieve good value in the construction of Stages 2 and 3 of the Highlands Trunk Sewer. Applications of tunnelling technology will likely be considered for rehabilitation or upgrading of urban utilities in the future because of its advantages in minimizing disruption in traffic flows and the environment, and minimizing other social costs associated with construction in urban centers.

PERSPECTIVES ON RESPONSIBILITIES FOR QUALITY CONTROL OF COMPACTION

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Introduction

I would like to start my discussion by saying that although the title of this paper clearly indicates "perspectives on RESPONSIBILITIES" will be discussed, I intend to speak only on the Consulting Engineer's perspective. I hope that upon completion of my talk, members of the audience will give their perspective on the responsibility for quality control of compaction.

I expect these other perspectives to come from other players involved in a typical underground water and sewer installation, i.e. the owner, the contractor, and the testing engineer/technician, [tester].

Review of the Contractual Arrangements

It was with reluctance that I delve into the domain of lawyers, for a number of reasons, not the least of which is that I am not an expert. For the purpose of providing some basic information on which to build my case however, I will briefly review some basic principles of the legal contract, which I am confident cannot be quarreled with to any great extent.

When an owner has decided that an underground utility is necessary or desired, he often enters into a contract with a consulting engineer to provide design and construction inspection services. [The case where an owner uses in-house engineering staff will not be discussed]. Within this contract the engineer has an obligation to the owner to **provide a design** of the underground utility according to prudent engineering principles currently in existence. The engineer further is obligated to satisfy himself that the **design concept is achieved,** and he will do this by performing some inspections and tests over the course of construction of the underground utility. The engineers inspections will also allow him to certify the **degree of completeness** of the construction, which will ultimately become a requirement under the contract between the owner and the contractor.

The tester can enter into a contract with the engineer or owner and/or the contractor. Looking only at the tester to owner/engineer contract for the moment, the tester is obligated to perform compaction test as outlined by various standards. The tester may in some instances have to certify that he **has performed** these tests in accordance with the set standards, as a requirement under the contract between the owner and the contractor.

After the owner has received the design, he enters into a contract with a contractor, to construct the underground utility according to design. The contractor is obligated to meet the **quality and quantity requirements** of the engineers design. During the course of construction, the contractor must set up his own **quality control program** to ensure that his obligations under the contract are met. The contractor also **warranties the work** done. The contractor should therefore enter into a contract with a tester to perform the same standardized compaction tests as a part of the overall quality control program.

Hypothetical Project From The Engineers Perspective

Tying the above rolls into a practical theoretical example, assume a municipality has determined that a 100 m section of existing 150 mm diam. watermain must be upgraded to 250 mm diam. The municipality, here-in called the owner, hires an engineer to:

- Design a new watermain,
- Prepare construction drawings, and specifications as a basis for construction contract,
- Provide resident engineering service including inspections to determine whether the design concept is achieved,
- The owner also hires a tester to make compaction tests and to certify that the tests were done to standard procedures,
- Provide certification of progress payments and completion.

The engineer proceeds to prepare plan/profile drawings, and specifies 250 mm PVC pipe, with imported sand bedding and pipe zone backfill, and re-use of existing granular material for backfill. He further specifies that the bedding and pipe zone backfill is to be compacted to 95% Standard Proctor Density (S.P.D.), and granular backfill is to be compacted to 97% S.P.D. After a typical public tendering process, the owner hires a contractor to install the new watermain as designed.

As the project proceeds to construction, dual testing procedures should be taking place. The engineer or the owner must hire a tester to take moisture/density tests and otherwise prepare to perform compaction tests so that the engineer can assure himself that the design concept is being achieved. It is not necessary for the engineer to have all the backfill tested, nor is there any set frequency of testing. The engineer simply must satisfy himself that the design concept is being achieved, bearing in mind changing conditions, weather and other circumstances. The contractor should retain a tester to take samples and otherwise prepare to perform compaction tests so that the contractor can ensure that each and every portion of the backfill is meeting the design requirements. This is important to the contractor because he must warrant proper performance of his work.

The progress payment certification process deserves some discussion at this point, as I have found it often becomes an issue in the roles/responsibility discussion. In our hypothetical example, the contractor has completed 456 m of watermain installation and backfill after one month, and claims payment for such during the first month. The engineer has had his tester perform random compaction tests to determine whether the bedding and backfill requirements are being met. Further the engineer has confirmed that in fact 456 m of watermain has been installed. The engineer therefore certifies the progress payment, and in so doing has done two things, and two things only:

1. He has satisfied himself that the design concept is being achieved, and
2. He has verified the degree of completeness of the construction.

This does not guarantee to the owner that **all** of the work has been performed to the quality level specified. Only the person who has performed the work can guarantee this performance, i.e. the contractor.

Upon completion of the construction, a similar certification procedure is required from the engineer. An additional certification is now required from the engineer's tester, as contractually agreed to, that the compaction tests have been performed and that they were done in accordance with standardized testing procedures. The contractor is obligated under the contract to remedy any defects in the work discovered in the guarantee period of one year. The contractor is still responsible for the cost of correcting defects in his work which occur after the guarantee period expires, but this obligation does not extend to making the repairs himself. The applicable statutory limitation on actions may eventually relieve the Contractor of that responsibility as well.

Some Common Misunderstandings (From the Engineer's Perspective)

1. **The Contractor relies on the Engineers testing for quality control.** The Engineers testing is done specifically to satisfy himself that the construction is done within the design concept. The testing does not require the frequency of tests necessary for a quality control program, nor should it, since it is the responsibility of the Contractor. To use the practical example, if at some point the Engineers testing indicates that the specified compaction is not achieved in the backfill, the Engineer's responsibility is to notify the Contractor that his construction procedure is not achieving the design concept and as a result, payment for this work cannot be certified. This is not a substitute for the ongoing quality control program of the Contractor. The Engineer does not supervise the Contractor's work nor does he take any responsibility for the quality of the Contractor's work.

> The failure of the professional to observe and prevent the contractor from improperly performing his contract should not provide an avenue for the contractor to say to the architect or engineer, 'because you failed in your contract with the owner to supervise me properly, you must in turn indemnify me or contribute to my burden of compensating the owner for my breach of contract.'[1]

2. **The Owner and/or Contractor assumes that certification of progress payments or completions is a guarantee of the quality of the work certified.**

The certification procedure done by the engineer only certifies the degree of completeness of the job, and general conformance to the design concept, not the absolute correctness of all the work.

> Providing that your responsibilities under your contract have included the provision of field services, you (the architect or engineer) must establish a reasonable inspection programme to enable you to issue the certificates required by the contract documents. This does not mean that you have to arrange for inspection of every square inch of the project, but the inspection programme must be designed to give you sufficient knowledge to be able to issue the certificates in question. You cannot certify either the quality or quantity of work without having seen at least a representative sample thereof.[2]

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[1] Excerpt from Construction Law Letter, July/August 1987, Case Comment "Duties of Architects & Engineers to Contractors Arising from Field Services."

[2] Loss Control Bulletin #82 Architects & Engineers – The Perils of Certification.

3. **The Owner requires the testing engineer to certify that compaction requirements have been met.**

The testing engineer is responsible for performing a test under rigid standards, and to inform his client of the results. The tester cannot guarantee that all compaction requirements have been met, because he has only tested where directed by others. What the tester can do is certify that he has done the tests and that the tests were done to the standards required. An example of the type of certification a tester should give follows:

> We hereby certify that we have observed the site during backfill compaction of underground and utility trenches, and have provided backfill testing services during the course of this work in accordance with accepted engineering practices, and based on the results obtained and the observations made, it is our opinion that the backfill compaction has been done in accordance with...(specifications.)[3]

Closing

In my experience in the municipal water and wastewater business, I have been involved in many arguments revolving around the role and responsibilities of the engineer in the inspector capacity, and the backfill compaction issue is a classic case. In a nutshell, I am saying that a person is only responsible for what he has agreed to do.

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[3] Wording for City of Edmonton testing engineer certification.

AERIAL COLOUR INFRARED PHOTOGRAPHY FOR LAGOON SEEPAGE DETECTION

INTRODUCTION

Municipal lagoons in Saskatchewan can be characterized most often as two-cell facilities. The first cell receives raw sewage influent and is considered a treatment cell. The second cell is usually referred to as a storage cell for treatment pond effluent. Lagoons normally operate at a depth of 0.9 metres to 1.2 metres deep, and are usually square or rectangular in shape. The average size of such facilities is around three hectares. [1]

Lagoon seepage may be defined as the migration of sewage effluent vertically and/or laterally out of the cells. Seepage may or may not be evident on the surface of the ground adjacent to a lagoon. In many cases seepage creates problems such as increasing soil salinity in surface soils, locally raising the water table, ponding of water outside the lagoon, and ultimately, loss of land productivity.

Lagoon seepage is not uncommon in Saskatchewan. In a recent survey Saskatchewan Environment and Public Safety and Sask Water identified approximately 250 municipal lagoons as having some type of operating problem. Many identified problems included seepage related concerns. These lagoons represent about half of all municipal wastewater treatment lagoons in the province. [2]

Seepage complaints have been investigated using a variety of techniques, from ground surveys to studying black and white aerial photographs.

Thorough ground surveys yield a great deal of information related to lagoon operations and seepage, though this approach is very time-consuming and labour intensive. Black and white aerial photographs have been used to complement the ground surveys and to reduce the number of detailed site visits. Existing photo records, however, are incomplete and may not reveal ground features in enough detail to be useful in interpreting lagoon operations.

Accordingly, planning and money has been directed toward creating an up-to-date aerial photography record of municipal lagoons. Colour infrared film has been specified as the main criterion for aerial photography work.

This paper discusses the application of aerial colour infrared photography to obtain information on lagoon seepage.

THE PROBLEM

The Environmental Management and Protection Act, 1984, stipulates that it is the department's mandate "to abate or prevent pollution". [3] This responsibility extends to ensuring that municipal wastewater works are operated in a specified manner as outlined in each community's permit for their treatment facility. To ensure compliance, the Department conducts tests, gathers data and may conduct investigations. Reliable, up to date information aids the Department in assessing treatment performance and documents any impacts on the environment. A good data base alerts the Department to treatment variations and operational difficulties associated with lagoon operations.

Annual compliance inspections, conducted by regional environment officers, provide a great deal of information related to lagoon operation and maintenance. These inspections also include seepage related concerns, such as the impact that a leaking lagoon may be having on adjacent land areas, or the extent of such seepage. Gathering information related to seepage typically requires an intensive site investigation by a community.

Progress in monitoring lagoon seepage and resolving seepage problems has been slow. A more efficient method of gathering information about lagoons led the Department and Sask Water to examine aerial colour infrared photography.

CHARACTERISTICS OF THE NEAR INFRARED WAVELENGTHS

Visible light, detectable by the human eye, is found at wavelengths between 0.4 and 0.7 microns in the electromagnetic spectrum. These wavelengths include colours that the eye sees as blue, green and red. Just beyond the visible wavelengths begins the infrared portion of the spectrum. The near infrared region, between 0.7 and 0.9 microns, can be made visible to the eye using specific photographic films that are sensitive to these longer wavelengths. Films used to detect near infrared may be black and white or colour.

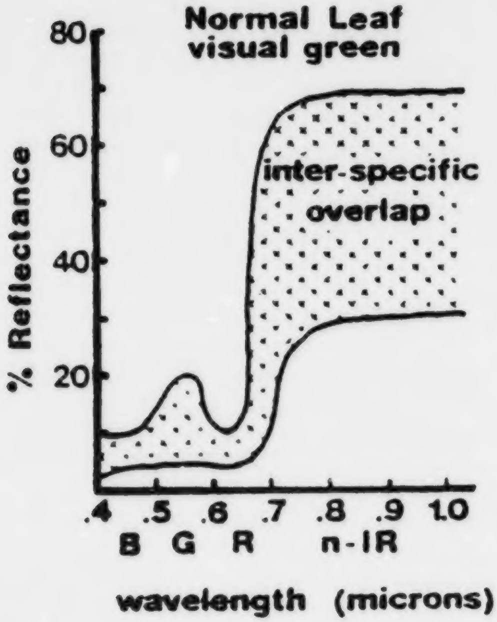
Colour infrared film can be defined as having the following characteristics:

> "Photographic film that is sensitive to energy in the in the visible and near infrared wavelengths, generally from 0.4 to 0.9 microns; usually used with a minus-blue (yellow) filter results in an effective film sensitivity of 0.5 to 0.9 microns; especially useful for detecting changes in the condition of the vegetative canopy that are often manifested in the near infrared region of the spectrum; colour infrared film is not sensitive in the thermal infrared region and therefore cannot be used as a heat sensitive detector." (4)

The use of a minus blue filter eliminates ultra-violet and blue wavelengths from colour infrared film. Removal of these wavelengths improves the contrast and resolution of the film. Different filters are required for various film types and photographic applications. In the case of lagoon aerial colour infrared photography, Kodak suggests a Wratten #12 filter to eliminate the blue portion of the visible spectrum while transmitting nearly all of the green and red portions. (5)

FIGURE I

Spectral Reflectance



Generalized spectrial reflectance patterns for a normal green leaf; inter-specific overlap indicates the wide variation and the overlapping of spectral reflectance patterns among various species of plants.

from Johannsen, C. J. and Sanders, J. L., Remote Sensing for Resource Management, Soil Conservation Society of America, Albeny, Iowa, 1982.

PURPOSE/USE/BENEFITS OF COLOUR INFRARED PHOTOGRAPHY

The usefulness of colour infrared film to detect subtle differences in vegetative cover is due to a property called spectral reflectance. Spectral reflectance represents the percent of incident light reflected by materials (including vegetation) as a "function of wavelength". Spectral reflectance increases in the photographic infrared region. This property makes it easy to discriminate vegetation types because vegetation has a wider range of reflective values in the photographic infrared region than in the green (normal colour) region. Figure 1 compares spectral reflectance of normal and infrared spectral regions. (note that the curves represent reflectance for a healthy green leaf).

Another useful feature of colour infrared film is early detection of plant stress. Wildman (1980) reported that "infrared film records changes in colour due to crop stress before the stress usually is apparent visually." (6) Healthy green plants are recorded as a red to magenta colour on colour infrared photographs. Similar plants experiencing growing problems, caused by salinity, disease, and other factors, characteristically have a lower infrared reflectance. This results initially in a darker red colour recorded on colour infrared film. This phenomenon is termed a "previsual" symptom of plant stress because it occurs days or weeks before a visual green colour change is evident. Table 1 presents "colour signatures" of both infrared and normal colour film, showing healthy and stressed vegetation.

In addition to being a good indicator of vegetative cover and plant stress, colour infrared photography can be effectively used to indicate drainage problems, soil moisture conditions and salinity. (7) Salinity, for example, may be revealed indirectly by plant stress and by salt/soil reflectance differences. Early effects of salinity affecting plants can be a dull red or purple colour recorded on colour infrared photographs. Complete absence of vegetative cover in saline areas may be indicated by white patches of bare ground or by a cyan colour on the soil surface.

Dramatic contrasts between water and land is another feature of using infrared photography. Clear water absorbs most infrared light resulting in a black or dark blue area on colour infrared photographs. Suspended materials in water, such as clay, may be indicated by lighter blue colours on photographs. Water logged soils may also be detected by a darker "signature", caused by the absorption of infrared energy. These contrasts can be useful in discerning the edges of slough areas, drainage ditches, and overgrown shore areas.

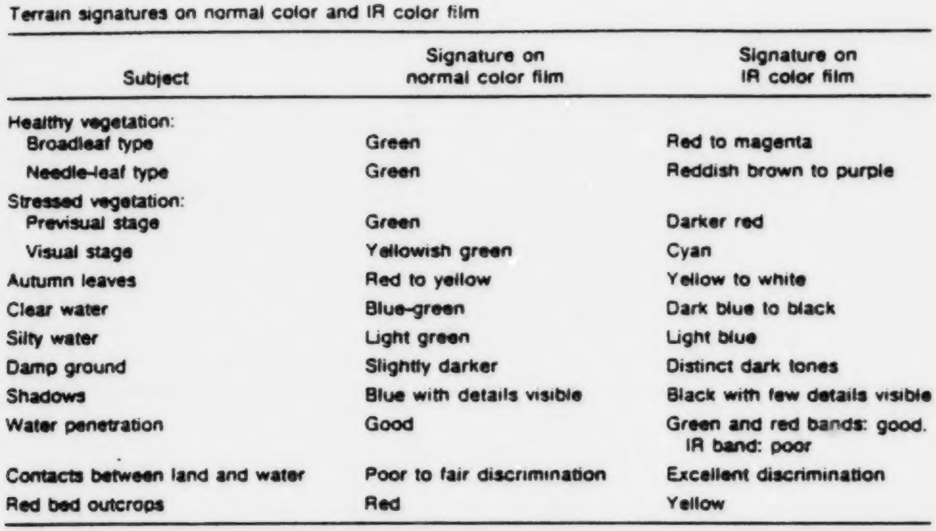
REMOTE SENSING

Remote sensing utilizing colour infrared film can be accomplished by different methods, primarily dependent upon the type of survey being undertaken and the degree of detail desired. Large scale studies often utilize infrared data gathered via satellite sources such as Landsat and SPOT (Systeme Pour l'Observation de la Terre).

With a spatial resolution of 80 m Landsat's Multispectral Scanner is suited for assessing rangelands, estimating crop areas, and monitoring long-term

TABLE I

Colour Signatures



Terrain signatures on normal color and IR color film

Subject	Signature on normal color film	Signature on IR color film
Healthy vegetation:		
Broadleaf type	Green	Red to magenta
Needle-leaf type	Green	Reddish brown to purple
Stressed vegetation:		
Previsual stage	Green	Darker red
Visual stage	Yellowish green	Cyan
Autumn leaves	Red to yellow	Yellow to white
Clear water	Blue-green	Dark blue to black
Silty water	Light green	Light blue
Damp ground	Slightly darker	Distinct dark tones
Shadows	Blue with details visible	Black with few details visible
Water penetration	Good	Green and red bands: good. IR band: poor
Contacts between land and water	Poor to fair discrimination	Excellent discrimination
Red bed outcrops	Red	Yellow

- from Sabins Jr., F.F. Remote Sensing Principles and Interpretation, W. H. Freeman Co., San Francisco, (1978), Page 44.

land-use trends. The satellite's Thematic Mapper has a better spatial resolution of 30 m, allowing more ground detail and sharp boundaries down to areas as small as two hectares. (8) (10)

The newer SPOT satellite carries two "High Resolution Visible (HRV)" imagers. The panchromatic sensor has a spatial resolution of 10m, while the colour HRV has 20 m resolution. While theoretically the SPOT sensors could record small wastewater treatment lagoons, details such as seepage boundaries and very localized vegetation stress could not be discerned. (9) (10)

For lagoon information gathering, aerial photography at low altitudes provides an opportunity to record much more information about specific sites than is currently possible via satellite imagery. The aerial vantage point is suited to recording details such as crop vigor, localized salinity, and subtle features such as water clarity . Flying altitude can be varied to accommodate different sized lagoon facilities to maintain consistent ground/lagoon cell coverage ratios on photographs. As well, many lagoons can be photographed in a relatively short period of time. This last point is important to ensure photographs are obtained in conjunction with sunny days and near cloudless conditions. Cloudy conditions tend to cause problems such as reduced incident light reflection from ground features and vegetation.

THE PROGRAM

In July, 1984, the Department undertook a preliminary study of lagoon seepage detection utilizing aerial colour infrared photography. The film was contained in a hand-held camera, used by a photographer while flying over pre-selected lagoon sites in a rented aircraft. In conjunction with the remotely sensed data, follow-up ground investigations were undertaken at several of the sites. The purpose of this study was to assess the overall effectiveness of remote sensing, utilizing colour infrared film, with the hope of establishing good documentation of seepage. This effort proved quite valuable and the Department decided to embark on a full scale project.

Since that time the Department and Sask Water have formalized lagoon photography and hired outside contractors to conduct the aerial photography work. Five contracts have been awarded over a five year period to photograph lagoon sites in most Environment Regions. To date over 340 municipal lagoons have been photographed. Figure 2 illustrates that lagoons have been photographed primarily in the Estevan, Regina, Melville, Watrous, Tisdale, and North Battleford regions. Regions still to be photographed include Kindersley, and Swift Current.

Some of the criteria specified in each contract includes: Kodak Aerochrome type 2443 , false colour infrared film; photograph lagoons perpendicular to the ground, parallel to the longitudinal axis; all lagoons to be photographed during July (maximum growth period for plants); and, take photographs under cloudless conditions whenever possible.

The finished product at the completion of each contract may be a complete set of 35mm colour slides and/or complete sets of 6"x8" colour prints. The Department prefers colour prints for easy reference. Colour slides on the

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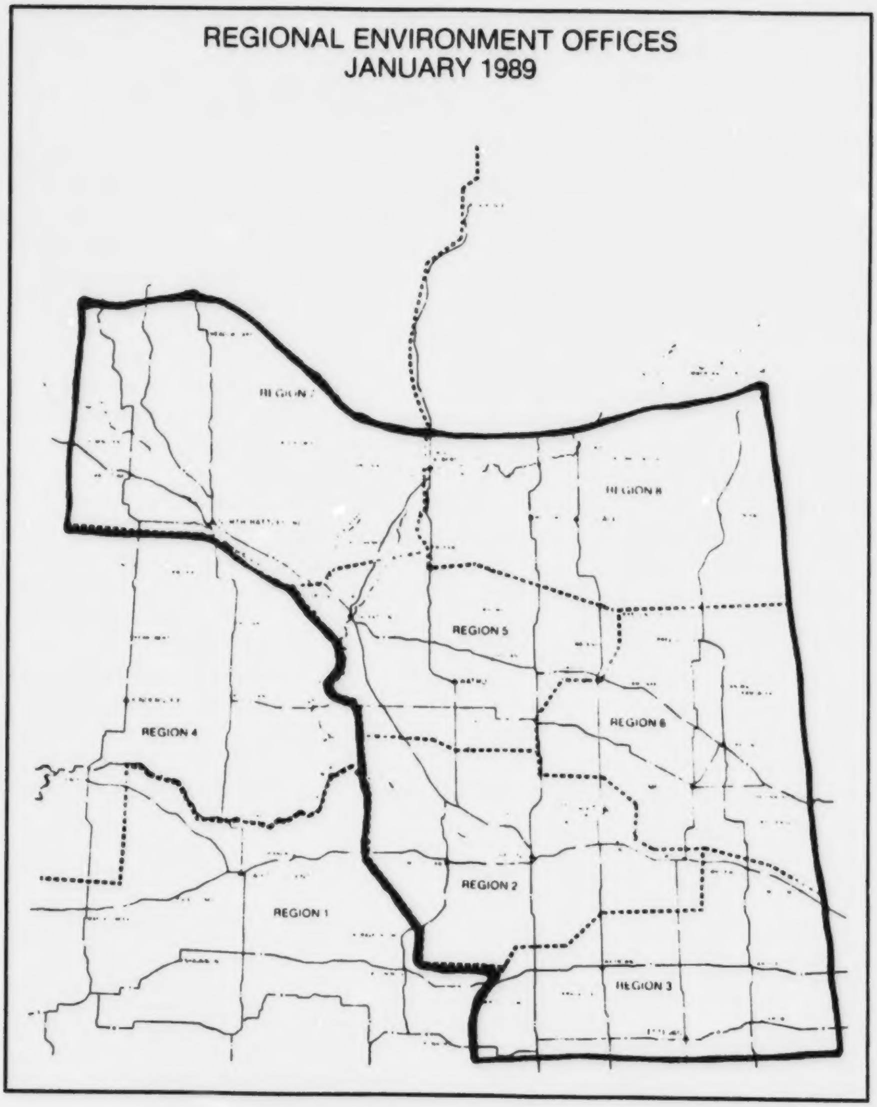
JANUARY 1989


REGION 7
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REGION 3

other hand are ideal for group viewing and are more easily stored than the prints. Either slides or prints can be reproduced for communities whose lagoons have been photographed as part of this program.

Finally, the complete lagoon colour infrared photo inventory is available to client users through the Central Surveys and Mapping Agency of the Saskatchewan Property Management Corporation.

CONCLUSIONS

The viewpoint afforded by aerial photographs expands the Department's and Sask Water's abilities to examine lagoon operations in considerable detail. By using colour infrared film, specific features are highlighted, which aid in revealing operational problems such as seepage.

Colour infrared photographs are helpful in evaluating known seepage problems and in revealing previously unknown seepage sites. This excellent source of up to date information on municipal lagoon operations is used extensively by Department staff. Photos are examined during permit reviews, and Regional Environment Officers use the photos to assess lagoon operations in their respective regions. Thus the Department is more effectively able to fulfill its mandate, "to abate or prevent pollution". Sask Water may also use the aerial photographs to investigate complaints, such as salinization caused by seepage, under the Water Corporation Act.

A very useful database is being created that is capturing lagoon operations at a certain point in time. It is planned that repeat photography of all lagoons will be undertaken once the initial survey is completed. Thus changes over time may be viewed.

Photographic images visually depict the impact of a lagoon problem. Both lagoon operators and municipal councils have been found to be much more receptive to the visual image, rather than to verbal discussions alone. The old adage "a picture is worth a thousand words" certainly applies to aerial colour infrared photography.

James C. Hoffman, July, 1990

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STREAMING CURRENT MONITOR-COAGULATION CONTROL IN WATER AND WASTE WATER PLANTS

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STREAMING CURRENT MONITOR FOR POTABLE WATER TREATMENT

Introduction

As quality standards on potable water tighten, the need for better control of coagulant dosages increases. Operators can no longer depend on periodic jar tests, or an occasional look at the flocculation section to properly run the plant. They must have a method of constantly knowing if coagulant dosages are correct to produce a consistently high quality water

Studies conducted by the U.S. Environmental Protection Agency, and others funded by the American Waterworks Research Foundation, as well as independent work done by municipalities, have shown that cysts such as Giardia and Cryptosporidium cannot be effectively removed without proper coagulant dosage. One recent study showed that a Dual Sand Filtration system was effective in removing greater than 99% of Giardia cysts, and greater than 98% of total coliform bacteria using proper chemical pretreatment. However if no chemical pretreatment was used, or if dosage was incorrect, removal was low and the system could not produce high quality water. Although Giardia is the most commonly known of the two cysts mentioned, evidence is that Cryptosporidium may be just as prevalent. Of 107 surface water samples collected during a recent survey 77 were positive for the presence of cryptosporidium cysts Our knowledge of Cryptosporidium is primarily due to the intensive research into the problem of Acquired Immune Deficiency Syndrome (AIDS). It turns out that a high percentage of AIDS victims contact Cryptosporidiosis, and never recover from the diarrhea. It is not the Cryptosporidiosis that kills AIDS victims, but it is one of the illnesses with which their immune system cannot cope. To protect against waterborne outbreaks of Cryptosporidium, filtration plants must rely on the same degree of treatment as required for Giardia. That is, minimizing turbidity breakthrough and optimizing filter performance.

There has been much public concern about lead contamination of drinking water. The USEPA Office of Drinking Water is working toward a treatment approach which assures that the water supplier treats the water to minimize corrosivity toward any lead it might encounter enroute to the consumers' tap. Precise coagulant control is critical to achieving this goal.

Clarification

The most critical time in the clarification process occurs within the first two minutes after the raw water enters the plant What the operator does, or doesn't do, in this short time span affects everything in the system to the household tap.

The results of coagulation control problems are well known:

- poor finished water quality
- Giardia and Cryptosporidium caused illness
- high chemical costs
- corrosion and fouling of distribution systems

- short filter runs
- high backwash costs
- excessive sludge
- increased pumping costs
- soluble aluminum in finished water

Every water plant operator wants to maintain good finished water quality. When everything is going smoothly, the plant pretty well runs itself. The operator's job has been described as one of many quiet days, interrupted occasionally by a few hours of mass panic. This panic is usually caused by such things as (1) coagulant pump failures, (2) heavy rainstorms causing widely swinging raw turbidities (3) instrument and controller malfunctions, etc. These upsets can cause any or all of the problems described above.

Coagulant Control Methods

Chemical dosages are determined and controlled by the plant operator. Experienced operators have learned how to cope with such factors as changing raw water turbidities and flow swings, but there are times when it is beyond their control to react quickly enough to maintain desired water quality. The general tendency is to overfeed chemicals as a "safety" measure. Jar tests, pilot filters, and zeta meters are some of the methods used to determine the required feed rates. However, operator experience and "eyeballing" are usually the deciding factors in actual plant operations. Jar testing has been the "standard test" for operators to determine the best coagulant dosage. Unfortunately most plants don't operate like jars, and many operators have discarded this method as a waste of time. Also. as mentioned before, when raw water turbidity is constant coagulant dosage can be set and left pretty much alone. The real need for running jar tests normally occurs when raw water turbidity is changing rapidly, which unfortunately, is when the operator doesn't have time to do them. Pilot filters have been used fairly successfully in some plants. At low to moderate raw water turbidities, they are helpful to the operator but tend to become almost useless at high turbidities. Also, the pilot filter results do not define if the water is being overdosed or underdosed when upsets occur. These systems require a fair amount of regular maintenance.

Zeta potential is a measure of the stability of a colloidal dispersion. Zeta meters generally provide a more precise measurement for determining correct dosages than jar tests or pilot filters However, this is a non-continuous sampling method, and requires good laboratory techniques to do it properly. Zeta potential measurements require the microscopic observation of floc particles moving through a capillary tube under an applied electric field. The particle velocities are converted to zeta potential and averaged to determine the "average zeta potential" of the system. High raw water turbidity makes the test difficult to run The test is also sensitive to water temperature changes and conductivity differences.

Streaming current is a measure of the electrokinetic charge (ionic and collidal) in a water sample. The streaming current amplitude and polarity is a function of the net charge (positive and negative). The sample is taken after coagulants have been added and thorough mixing achieved. Since the SCM samples continuously, there is no possibility for operator error in sampling or testing. The device incorporates best features of the other three control methods without the disadvantages inherent in each.

Streaming Current Monitor

The Streaming Current Monitor (Figure 1) is an online instrument that enables the treatment plant operator to know precisely, and at all times, the optimum coagulant dosage The SCM responds to changes in raw water characteristics (turbidity, pH, color, etc.) and flow rates and allows the operator to make the necessary adjustments It also provides an immediate alarm if chemical feed is interrupted for any reason. The SCM works equally well with inorganic coagulants (alum, Ferric compounds, etc.) and all polymers that possess positive or negative charge As any plant operator can readily testify, plant upsets almost never occur during the daytime. They usually happen at night when there is a minimum of help and he can t see the basins very well.

FIGURE 1

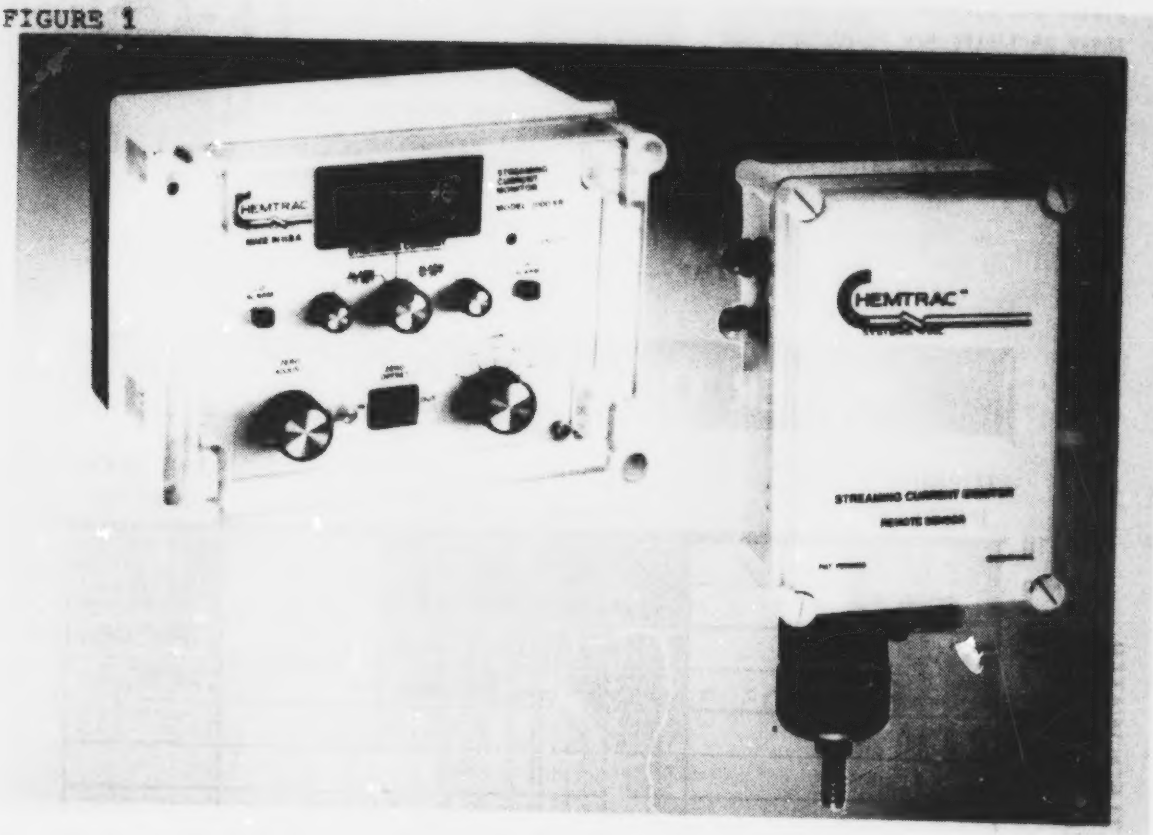


Figure 2 is a representation of upsets that occur causing any or all of the problems previously stated. The SCM enables the operator to know almost immediately that an upset has occurred, rather than waiting several hours to see problems in the finished water. Figure 3 shows how the SCM fits into the treatment scheme of a water plant. The unit works in direct filtration plants as well as the conventional process shown. The sample must be taken at a point where uniform distribution and mixing of coagulant is obtained for all flow rates. If uniform distribution and mixing is not obtained at a selected sample point, the SCM reading will oscillate. Proper sample selection is critical to attaining maximum benefit from the SCM.

The SCM is a charge measuring device. The charge measured is the net ionic and colloidal surface charge (positive and negative) in the sample being tested. The treated sample flows into the cell where a small portion is drawn into the bore during the piston upstroke and expelled on the downstroke (Fig. 4) Particles in the sample are temporarily immobilized on the piston and cylinder surfaces. As the water is moved back and forth, mobile charges surrounding these particles are moved downstream past the electrodes. This reciprocating movement of like charges causes an alternating current to be generated; this current is defined as the "streaming current". Anything that affects the charge condition of the sample will affect the streaming current value. These include turbidity, pH, conductivity, chemical feed, etc.

FIGURE 2

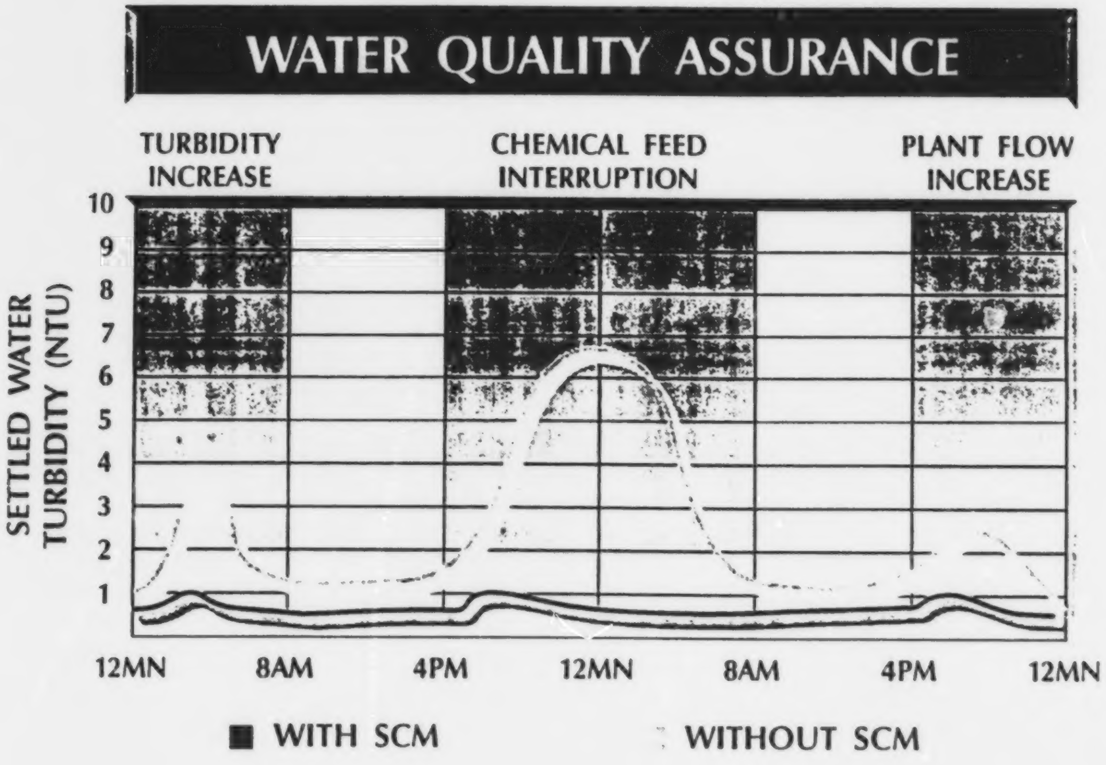


FIGURE 3
SCM Typical Installation

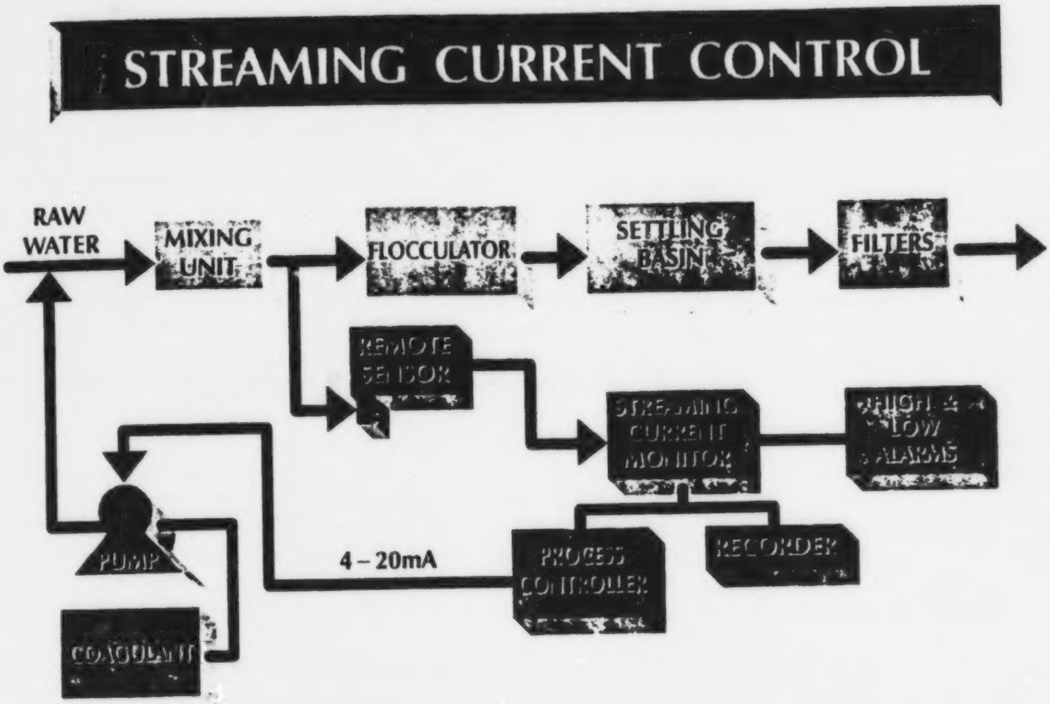
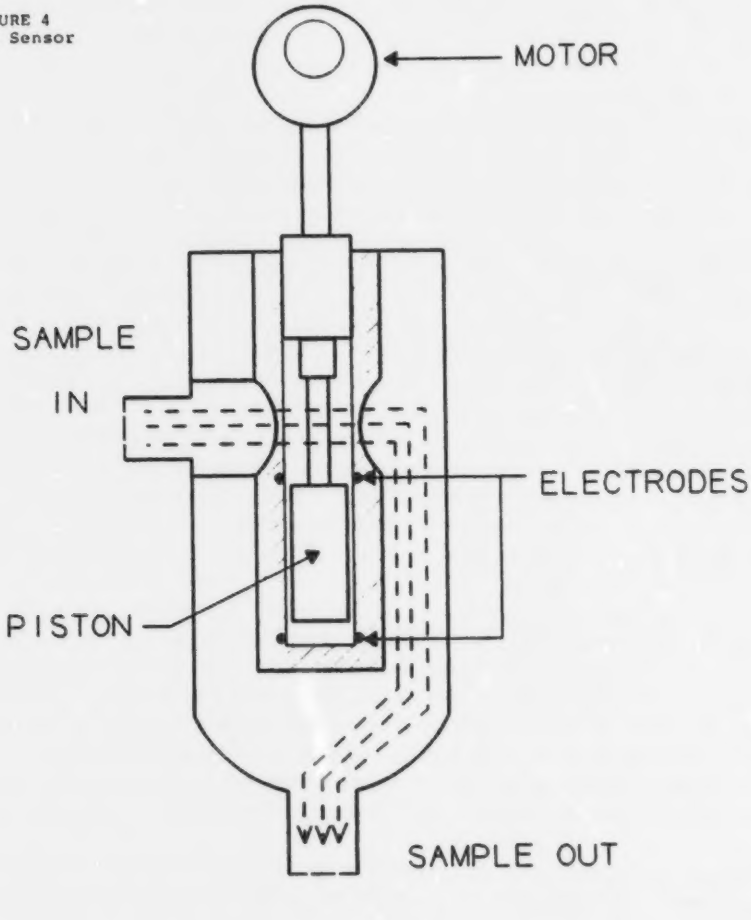

STREAMING CURRENT CONTROL
RAW WATER
MIXING UNIT
FLOCCULATOR
SETTLING BASIN
FILTERS
REMOTE SENSOR
STREAMING CURRENT MONITOR
HIGH & LOW ALARMS
PUMP
4 – 20mA
PROCESS CONTROLLER
RECORDER
COAGULANT

FIGURE 4
SCM Sensor


MOTOR
SAMPLE IN
ELECTRODES
PISTON
SAMPLE OUT

Coagulant Optimization Using SCM

Although most plants do the job of producing good. clean water, virtually everyone overfeeds coagulants as a "safety factor " This overfeed may range from 10% to 50% or higher. To optimize his system, the operator simply adjusts coagulant dosage and observes what reading on the SCM corresponds with the lowest feed rate that produces acceptable water quality. This is his "set point." This set point will usually remain very close to the same value during periods of high raw water turbidity. He simply adjusts coagulant feed to maintain this value. The SCM produces a 4-20 milliamp signal that can be used to automatically control the coagulant feed. The SCM also has relays for High and Low alarms that can be set to alert the operator to treatment upsets.

Waterborne Diseases

There is a growing concern over outbreaks of giardiasis and Cryptosporidiosis--illnesses caused by single-cell organisms found in many surface waters. These diseases were once thought to be caused only by aquatic animals such as beaver or muskrat. It is now known that almost all warm-blooded animals are implicated.

The treatment techniques for removing Giardia and Cryptosporidium cysts from drinking water do not differ from conventional means The multiple barrier method of coagulation, flocculation, and sedimentation, followed by filtration and disinfection, does a very effective job of removing these organisms. However, studies have shown that as settled water turbidities increase, so do cyst levels. This problem is usually caused by improperly controlled coagulant feed, or complete loss of feed from some length of time.

In the case of direct filtration systems, proper coqgulant feed is absolutely critical in preventing cyst transport. Studies have snown that is is virtually impossible to remove these cysts without coagulant. In many direct filtration systems, the time lapse between the point of goagulant injection and the filter inlet is very short. The SCM can provide the immediate alert needed by the operator to take corrective action and minimize a very serious problem.

Aluminum Carrythrough

Aluminum usually enters the distribution system as a result of poor coagulation, alum overfeed, and/or poor pH control in the flash mix. This is especially true in raw waters with low alkalinity where lime or caustic is used to adjust pH at coagulation. Alum has a fairly wide pH range in which good flocculation and settling occurs, but there is a specific pH at thich the minimum amount of aluminum will carry through the filters and into the distribution system. The SCM, in conjunction with pH control will minimize aluminum carrythrough.

Corrosion and Fouling of Distribution Systems

The detrimental effects of postprecipitation have been known for years. Aluminum hydroxide can be deposited on any type of pipe lining. Even a thin coating will reduce water throughput capacity and increase pumping costs. Deposits in lines accelerate corrosion mechanisms, and can even contribute to microbiological growth. Microorganisms that are not actually chlorine-resistant have been isolated in pipe samples. This is probably caused by the protective effect of tubercles and the localized high chlorine demand. Corrosion control programs using phosphates and/or zinc are adversely affected by aluminum carrythrough. Feed rates of these inhibitors must be increased to maintain desired corrosion protection.

Most treatment plants do not routinely test for Alum in the finished water. Although there is as yet no EPA requirement to report aluminum content, many treatment plants are preparing to begin to test for it routinely. Plants that do run this test have found significant reductions in finished water aluminum when the SCM was put into service and the alum dosages minimized. One must remember that even a small overdose of 1 to 2 mg/l can significantly increase aluminum carrythrough.

System Analysis Capability

The SCM has proven to be an invaluable tool in helping solve many water treatment problems. By being able to continuously measure the net electrokinetic charge at any point in the system, the effect of erratic chemical feeders, variations in coagulant strength, poor chemical mixing, distribution problems, short circuiting, etc., can be seen and corrected. The net result is a higher quality water going to the customer. It also results in less plant maintenance and operational problems, and improves overall plant efficiency.

Conclusion

Long term, in-plant use of the SCM to control the coagulation process has provided the following results:

- Consistent, high quality finished water
- Reduced total chemical costs
- Reduced soluble aluminum carrythrough
- Reduced distribution line fouling and corrosion
- Increased length of filter runs
- Less sludge generated
- Less sludge disposal costs

The SCM has been evaluated in several water treatment plants alongside pilot filters and zeta meters, and has been chosen as the coagulation control system for the following reasons:

- The SCM continuously generates a signal that shows the operator if he or she is overdosing or underdosing.
- The SCM doesn't require backwash, media replacement or calibration.
- The SCM performs accurately and reliably at all turbidity levels normally encountered.
- The signal generated can be used to automatically control coagulant feed.
- High and low chemical feed alarms minimize plant upset.

OPTIMIZING SLUDGE DEWATERING COSTS USING THE STREAMING CURRENT MONITOR

Introduction

The ideal situation in a waste treatment system would be "steady state" conditions, i.e., constant flow, constant solids concentration (and type), constant chemical feed, etc. This is very rarely the case. As solids holding vessels are filled and pumped out, flow rates and solids concentrations change. Polymer make-up may not always be the same, nor pump rates consistent. It is impossible for the equipment operator to keep up with these changes and adjust flocculant dosages accordingly. Therefore, the tendency is to overfeed to smooth out the bumps. This overfeed can amount to a sizeable amount of money on a yearly basis.

Polymer costs in many sludge dewatering operations is a major concern. Although new and improved methods are continually being developed to dewater sludge, most equipment still requires polymer to attain desired throughput and cake dryness. If improvement in cake dryness can be gained through optimizing polymer dosage, or if desired cake dryness can be obtained with less polymer, the savings in large plants, can be enormous, especially when the cake is being incinerated or hauled to landfill. Polymer optimization is only one parameter that contributes to overall cost effectiveness. However, it is an area that has a major impact, and one where there is much room for improvement. The SCM can be a very effective tool in helping plants improve overall cost effectiveness.

Sludge Dewatering Process

Sludge dewatering simply means increasing the percent solids of a slurry by a mechanical process. Polymers are typically used to improve this process by increasing cake dryness and/or sludge throughput.

Although per ton polymer costs may vary widely, it isn't unusual for a medium size municipal waste treatment plant to spend $50,000 per year. On a monetary basis, polymer feed rates on belt presses are manually controlled by the plant operator based on visual observation of sludge "formation" on the belt, cake dryness and filtrate quality. On centrifuges, there isn't any visible indication of momentary machine performance except cake and filtrate appearance. These are very subjective methods. Actual polymer dosages may vary several percent due to different operator's opinions. Optimum dosage is generally defined as that which gives the highest cake solids, maximum throughput rate cleanest filtrate/pressate minimum maintenance, or some combination of these.

It is an axiom that no two sludges are the same. Polymer requirements and operating techniques may be different on individual dewatering machines in the same plant. It is a fairly normal proceedure for the operator to set feed rates above the optimum to handle swings in sludge flow or polymer demand.

Streaming Current Control

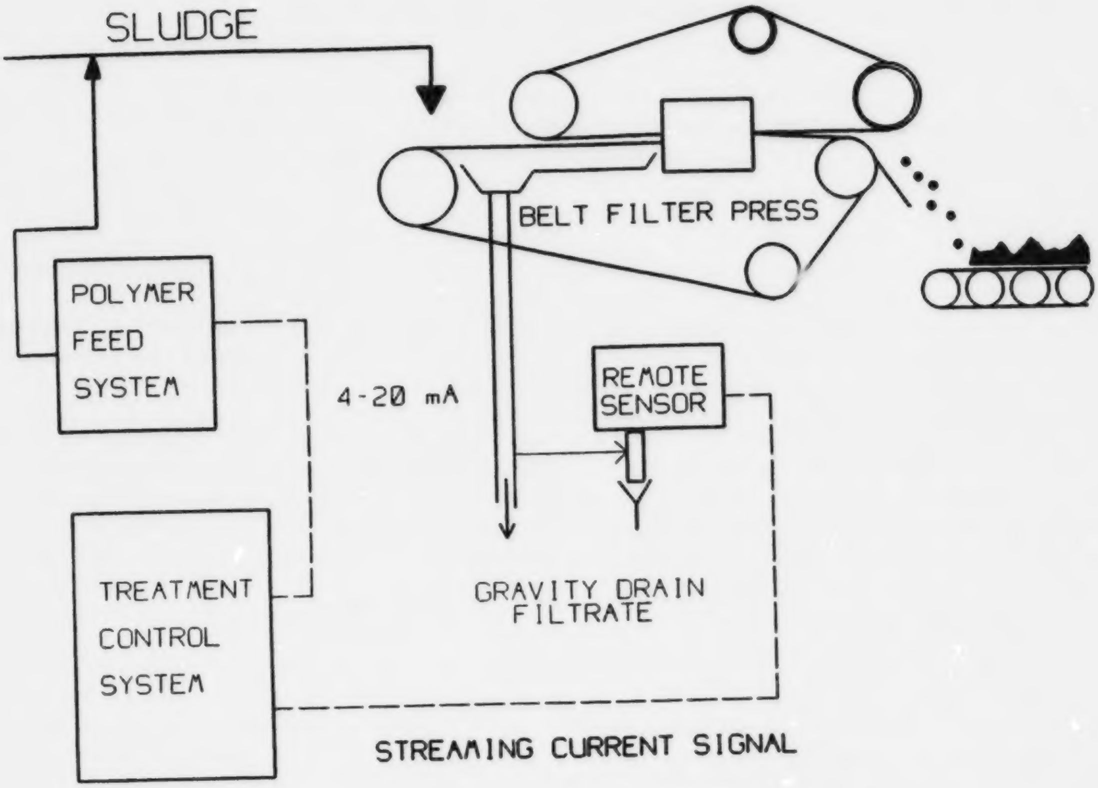
Using polymers for sludge flocculation is a combination of charge neutralization and mechanical bridging. Most municipal sludges that have undergone anaerobic or aerobic digestion possess a high negative charge. A positively charged chemical is required to flocculate particles together and improve dewatering efficiency. The trick is to feed just the right amount. Underfeed may cause: (1) a decrease in cake solids, (2) slowing down belt speed and sludge throughput, (3) increased filtrate solids, (4) increased hauling or incineration costs. Overfeed may cause: (1) high chemical costs, (2) belt plugging (3) decrease in cake solids.

As previously mentioned, the plant operator has had to depend on periodic visual observation to judge polymer requirements. The streaming current monitor offers a continuous, accurate method of not only monitoring but continuously controlling the dosage.

The SCM responds to changes in raw sludge flow, solids concentration and flow rates, and allows the operator to make the necessary adjustments. It also provides an immediate alarm if chemical feed is interrupted for any reason. The SCM works equally well with inorganic coagulants (allum, ferric compounds etc.) and all polymers that posses positive or negative charge.

On a belt filter press, (Figure 1) the sample is taken from the gravity section filtrate. This sample is usually clean enough to be passed through the the SCM sample cell with little fouling problem. Some installations may require a course strainer to prevent large or fibrous materials from plugging sample pumps or transfer lines. The centrate is sampled on centrifuges.

FIGURE 1


SLUDGE
BELT FILTER PRESS
POLYMER FEED SYSTEM
4-20 mA
REMOTE SENSOR
TREATMENT CONTROL SYSTEM
GRAVITY DRAIN FILTRATE
STREAMING CURRENT SIGNAL

Polymer Optimization Process

The polymer dosage is optimized by making incremental changes in the feed rate to attain desired (1) sludge formation on the belt, (2) cake thickness, (3) cake release from belt onto conveyor, (4) filtrate clarity, (5) cake dryness. As previously mentioned, it is somewhat of a balancing act to integrate all operating parameters to obtain optimum system performance. The filtrate charge is effected by sludge flow, sludge solids percentage and type, and polymer dosage. Therefore, the streaming current value is a very important input in the control program. Computerized operator programs are being developed by belt filter press manufacturers that incorporate streaming current measurement in the control scheme.

Conclusion

The potential is great for cost savings in sludge dewatering applications due to the high cost of polymer. Long term success in hundreds of potable and high purity water systems has shown the Streaming Current Monitor to be an extremely reliable device.

VOLATILE HYDROCARBONS IN DISTRIBUTION SYSTEMS

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INTRODUCTION

Benzene, toluene, ethylbenzene and the three xylene isomers (*o*-, *m*- and *p*-xylene) are all used extensively as solvents, paint thinners and raw materials for synthesis of plastics (1-3). These volatile organic compounds (VOCs) are also common components of gasoline (4); benzene is present in gasoline at concentrations of approximately 1 to 2 percent (5). As a result, there are many possible ways to contaminate both source waters before treatment as well as potable water in distribution systems. Benzene, for example, could be introduced into source water from industrial effluents or atmospheric pollution, and into finished drinking water by leaching from plastic products. Leaking underground storage tanks could contaminate surrounding soil and ground water with gasoline. Consequently, any disruption in distribution lines could result in contamination of potable water with VOCs. Penetration of plastic distribution pipes with VOCs from surrounding soils is also possible.

Canadian Drinking Water Quality Guidelines (6) specify a maximum acceptable concentration of 5 ug/L for benzene. Maximum acceptable concentration limits were not set for alkyl benzenes because they impart an objectionable taste and odour to potable water at concentrations well below those considered harmful. The aesthetic objectives (6) for toluene, ethylbenzene and xylene isomers are ≤ 24 ug/L, ≤ 2.4 ug/L and ≤ 300 ug/L, respectively. Unlike the disinfection byproduct trihalomethanes (THMs), most utilities do not monitored VOCs regularly in drinking water. In fact, very limited information (dating back almost 10 years) is available on the levels of VOCs in Canadian drinking water supplies. In 1982 Otson *et al.* (7, 8) analyzed VOCs in samples from 30 drinking water treatment facilities across Canada. At that time, benzene was detected in 60% of potable water samples with mean concentrations ranging from 1 to 3 ug/L. These concentration levels for benzene were menacingly close to the maximum acceptable concentration of 5 ug/L and certainly warrant greater monitoring efforts. Other VOCs were also frequently detected in source waters with toluene, ethylbenzene and xylenes present in 45%, 25% and 12% of samples, respectively. Toluene was found in finished water at 2 ug/L, while ethylbenzene and *m*-xylene were detected at less than 1 ug/L.

In 1982, Calgary established a monitoring program for both disinfection byproducts (THMs and related compounds) as well as VOCs in potable water. Over the past 8 years, the accumulating data have established the normal range of expected concentration levels for these two compound groups in finished water leaving the two treatment plants and in the distribution system. Benzene, toluene, ethylbenzene or the xylenes have never been detected at concentrations exceeding the dynamic headspace GC-PID method 0.5 ug/L quantitation limit.

This background information was invaluable in December 1989 when the Waterworks Laboratory received a complaint from a homeowner in a new subdivision. The customer described his water as having a distinct paint smell. Within 4 hours of receiving the sample, the VOCs present were identified and quantitated with concentrations totalling more than 500 ug/L. Residents in the affected block of homes were immediately informed, asked not to use their tap water and supplied with a portable water wagon. It was later determined that the xylenes and other aromatic hydrocarbons had been released into the water from the epoxy lining of a recently installed water supply main.

It was fortuitous that a rapid, automated method for VOCs analysis was already established at the Waterworks Laboratory and that some information regarding typical concentration levels of VOCs in areas of Calgary with different distribution pipe materials already existed. This paper describes the dynamic headspace method for sampling VOCs as it compares to the more commonly used Purge and Trap (P&T) procedure (9,10) and also summarizes the investigation of a drinking water VOCs contamination incident.

MATERIALS AND METHODS

Gas Chromatograph. A Hewlett-Packard Model 5890 Gas Chromatograph was used equipped with Flame Ionization Detector, 15 mCi 63 Ni Electron Capture Detector, HNU Model 52-02A PID with 10.2eV lamp (HNU Systems Inc. Newton, MA) and Hewlett-Packard Model 3390 and Model 3396 integrators. DB5 (30 m x 0.530 mm i.d. fused silica) capillary column (1.5 um film thickness 5% phenyl methyl silicone bonded phase, J and W Scientific (Chromatog. Specialties, Brockville, Ont.) operating conditions were: Temperature (°C)- Injector, 225°; Column programming from 35° (hold 4 min) to 150° at 10°/min and hold 6.5 min; Detector, 200°. PID Lamp intensity - 50%. Carrier gas-Helium UHP at 35 kPa (provided a carrier flow of 8 mL/min); split vent flow, 6 mL/min; sample transfer flow from sampler, 7 mL/min. For each sample injection the split ratio was, therefore, 8:21. PID makeup gas: 20 mL/min Helium UHP.

Automatic Dynamic Headspace Sampler. Hewlett-Packard Model 19395 Dynamic Head Space Analyzer. Sealed headspace sample vials were placed on the 24-sample capacity autosampler tray immersed in an oil bath (70°C) and were equilibrated for 40 min prior to analysis. During analysis the sampler needle pierced the Teflon faced silicone septum (Hewlett Packard Cat. No. 9301-0719) and filled a 1 mL gas-tight loop with a headspace sample. The transfer lines to the GC injection port were maintained at 75°C to prevent condensation of the sample.

Gas Chromatograph-Mass Spectrometer. The presence of xylene, ethylbenzene and other compounds was confirmed by GC-Mass Spectroscopy (GC-MS). The headspace sampler could easily be moved from the GC-FID to the GC-MS. A Hewlett Packard Model 5993 quadrupole electron impact mass spectrometer coupled to a Model 1000 data system was used with run conditions identical to those described for the gas chromatograph.

Preparation of samples. Headspace sample vials (20 mL glass vials, Hewlett Packard Cat. No. 9301-0716) were pretreated by adding 2 mg of sodium thiosulphate to dechlorinate treated water samples. With minimum agitation, 15 mL of water sample was added and the vial was immediately sealed using a Teflon faced silicone septum and aluminum crimp caps (20 mm aluminum crimp caps, Hewlett Packard Cat. No. 9301-0721). Samples were analyzed as soon as possible after collection, but if necessary could be stored inverted and at 4°C for up to 14 days with little change in compound concentrations.

Standards and blanks. Stock standard solutions of benzene, toluene, ethylbenzene and the xylenes were prepared from ACS grade reagents (BDH, Toronto, Ont.) at 10 ug/mL in methanol. These standards were stored in sealed Reactivials with Mininert caps (Chromatog. Specialties, Brockville, Ont.) to prevent any loss of the volatile standard compounds or the solvent. Fresh working standards containing 10 ug/L of each compound were prepared prior to analysis of each batch of samples by adding 50 uL of the stock standard into 50.0 mL of reagent water (MilliQ system with OrganicQ and activated carbon cartridges) using a microlitre syringe. Other standard concentrations were prepared similarly. The solutions were gently mixed and 15 mL volumes were immediately transferred into headspace vials. The GC-PID was calibrated using the 10 ug/L standard external calibration method. MilliQ reagent water was used as the blank.

RESULTS AND DISCUSSION

COMPARISON OF METHODS FOR ANALYSIS OF VOCs - DYNAMIC HEADSPACE GC-PID VERSUS P & T GC-PID.

At the City of Calgary Waterworks Laboratory, the automated dynamic headspace GC-PID method has been used successfully for quantitative analysis of compounds ranging from benzene to naphthalene with minimal loss of sensitivity for higher molecular weight compounds. The HNU PID has good sensitivity, minimal background noise, linear response and is simple to operate. Figure 1 shows a typical chromatograph obtained for the analysis of a 5 ug/L standard solution of VOCs. Dynamic headspace GC-PID analysis for VOCs has a wide linear range. A calibration graph over the range of 2 to 100 ug/L *p*-xylene is shown in Figure 2. Higher concentration samples can be analyzed by increasing the GC split ratio.

FIGURE 1

Chromatograph [dynamic headspace GC-PID] of a standard mixture [15 mL water sample] containing 5 ug/L each of: A] Benzene, B] Trichloroethane, C] Toluene, D] *p*-Xylene E] *o*-Xylene, F] *t*-Butylbenzene, G] *p*-Isopropylmethylbenzene and H] *m*-Dichlorobenzene. [A blank chromatograph obtained by analyzing reagent water is shown below the sample chromatograph.]

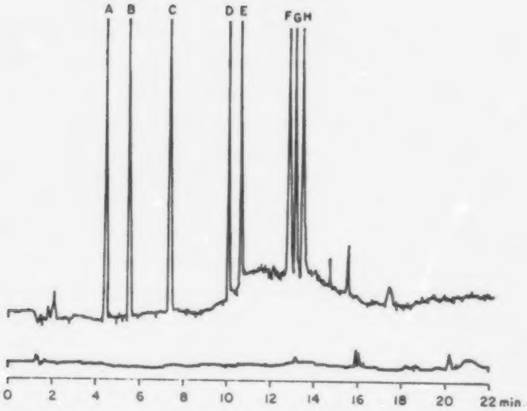
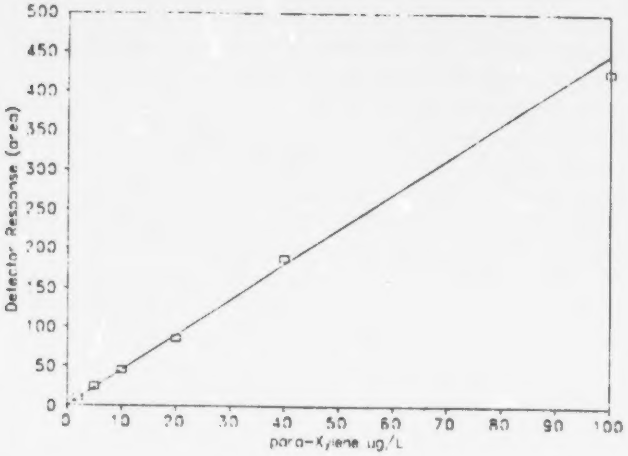


FIGURE 2

Calibration graph for p-Xylene analysis by dynamic headspace GC-PID in the concentration range 2 to 100 ug/L. (Correlation coefficient, r=1.01, slope=4.22,y-intercept=2.77).



Other well-established analytical methods for volatile hydrocarbons most often use P&T to concentrate the volatile sample constituents and improve method sensitivity. The U.S. Environmental Protection Agency determined that the practical quantitation limit for benzene in water by P&T GC-PID (based on the ability of laboratories to measure benzene within reasonable limits of precision and accuracy) was 5 ug/L (9). The detection limit for toluene, ethylbenzene and the xylene isomers by P&T GC-PID was reported (10) to be 0.02 ug/L, while the detection limit for confirmation of each of these compounds by P&T GC-MS was at ten times higher concentration (0.2 ug/L). In spite of the low reported detection limits for toluene, ethylbenzene and the xylene isomers by P&T procedures, Supporting Documentation to the Canadian Drinking Water Quality Guidelines (4) reported that the practical quantitation limit (based on the ability of laboratories to measure these compounds within reasonable limits of precision and accuracy) was probable also about 5 ug/L, as for benzene. Sampling by P&T or dynamic headspace method, therefore, give comparable practical quantitation limits. The dynamic headspace GC-PID method used at the City of Calgary Waterworks Laboratory has excellent reproducibility, with less than 3% relative standard deviation for benzene, toluene, ethylbenzene and the xylenes at a concentration of 5 ug/L.

In addition to being selective for aromatic compounds with greater sensitivity than a flame ionization detector, the PID is also a nondestructive detector. This attribute would allow sequential determination of volatile hydrocarbons using the PID followed by the determination of volatile chlorinated compounds using an electron capture detector. Using this configuration, it could be possible to analyze a single sample simultaneously for both VOCs and THMs or other halogenated volatile organic compounds.

While the P&T sampling method offers better "theoretical" detection limits through more efficient sample transfer, it is also more difficult to automate than dynamic headspace sampling. Automated P&T systems can suffer from sample carry over in the purging vessel and peak ghosting due to incomplete desorption from trap materials. The dynamic headspace sampling with GC-PID analysis method has proven to be a reliable, automated method for analyzing volatile hydrocarbons in drinking water. Sample manipulation is minimized since samples can be collected, stored and analyzed in the same container. This reduces the possibility of artifacts or false positive samples.

DISTRIBUTION SYSTEM CONTAMINATION INCIDENT INVESTIGATION

VOCs analysis of source, finished and distribution system water has been conducted at the City of Calgary Waterworks Laboratory for over 8 years. Typically in Calgary's finished water no VOCs are found at levels above the detection limit of 0.5 ug/L. In response to a water quality complaint in December 1989, samples were collected for analysis of VOCs, halogenated volatile organic compounds and extractable base-neutral organic compounds from (Figure 3) the complainant's house, a neighboring house, feeder mains in the zone and from the main supplying the zone. The samples from the two houses, contained each of the three xylene isomers at concentrations over 200 ug/L as well as 5 to 100 ug/L each of benzene, toluene and ethylbenzene. Many unresolved or unidentified aromatic compounds were also present in all of the initial samples collected from the homes. A typical GC-PID chromatograph is shown in Figure 4.

FIGURE 3

Site map indicating pipe materials and occupied houses affected by VOCs contamination.

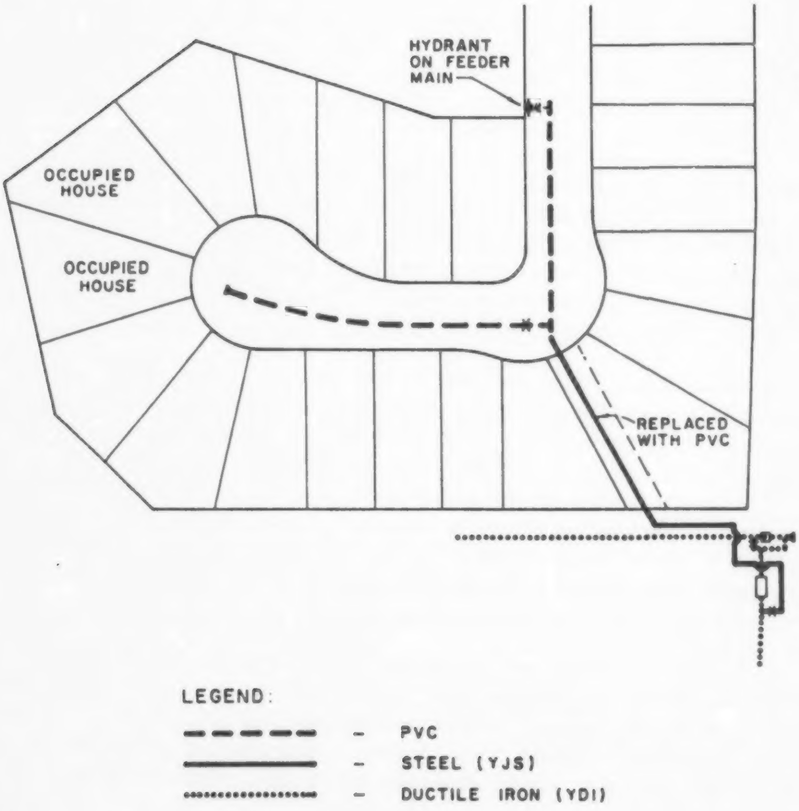
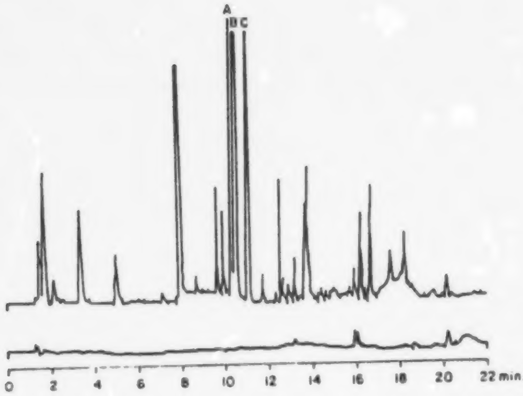


FIGURE 4

Chromatograph (dynamic headspace GC-PID) of water sample from collected from the contaminated YJS main contained: A] m-Xylene (45 ug/L), B] p-Xylene (90 ug/L), C] o-Xylene (47 ug/L) and other unidentified aromatic compounds. (A blank chromatograph obtained by analyzing reagent water is shown below the sample chromatograph.)



Following initial sample collection, the main was flushed for 2 hours through the nearest fire hydrant in an attempt to remove contaminated water from the distribution system. Following flushing, a second set of samples were collected from the houses and from two hydrants. It was anticipated that the initial flushing would remove all traces of solvent or at least greatly attenuate concentrations present. However, samples from both of the houses still contained 100-130 ug/L xylene (all isomers) but no detectable concentrations of other VOCs.

Contamination by VOCs was limited to one supply main. Since the area was still under development, there were only two occupied houses being supplied by the affected main. The feeder main was dead ended directly in front of these two houses making hydraulic isolation of the pipe straight forward but complicating sampling and flushing procedures. The feeder main bringing water to the area was used as a reference location throughout the incident; no VOCs were ever detected. Numerous samples from adjacent areas, other supply mains and the water treatment plants also contained no trace of VOCs. Further, dynamic headspace GC analysis for volatile halogenated compounds using an Electron Capture Detector identified no unusual compounds.

Less than 4 hours after receiving the sample in the Waterworks Laboratory, the two homeowners were advised not to drink water from their homes and an emergency water supply wagon was provided. The following day another set of samples was collected before, during and after an extensive hydrant flushing. Typical total xylene concentrations from the two houses were 15 ug/L before flushing, 100 ug/L during flushing and 5 ug/L after flushing. To provide more representative sampling and to assist in main flushing, the water services for unfinished lots in the immediate area were fitted with sampling taps and a 50 mm flushing nozzle was installed on the dead end of the feeder main. Three days later more samples were collected after extensive hydrant flushing and less than 5 ug/L xylenes were detected. While further investigations continued, normal water service was returned to the two homes through an alternate main.

To determine if the xylene levels would remain low without the aid of flushing, a sample was collected from the contaminated main after flushing was suspended for six hours. This sample had a xylene concentration of 27 ug/L. Repeated sampling during the next two weeks gave similar results. By flushing the feeder main, xylene concentrations could be temporarily reduced but soon after flushing was suspended, levels would again rise to 20 - 50 ug/L. Flushing was not effective in permanently removing the contamination source so the possibility of soil contamination near the water pipes and the pipe materials themselves were then investigated.

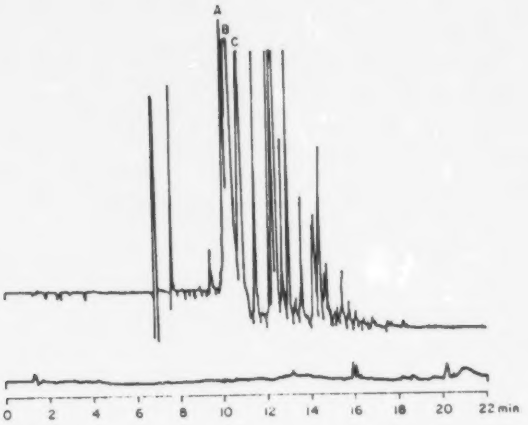
Many different materials and sizes of pipe were installed in this area of the distribution system, ranging from 150 mm polyvinyl chloride (PVC) to 400 mm steel and ductile iron pipes up to 900 mm concrete mains. One hypothesis was that contaminated soil had inadvertantly been used as backfill and solvent was leaching through the PVC pipe. This was, in retrospect, an unlikely scenario since the PVC pipe was intact and was pressurized. The other hypothesis was that a section of the pipe or a control element was contaminated and that this was the source of contamination.

The faulty pipe was narrowed down to three sections of main: one PVC, one epoxy-lined yellow ductile iron (YDI) and one epoxy-lined steel (Yellow Jacket Steel, YJS). Sampling showed the YDI section of pipe was free of contamination. Samples were then collected throughout Calgary from dead end mains where construction materials were PVC or YJS. No xylene or other volatile hydrocarbons were detected from any of the PVC samples but several water samples from one section YJS pipe had total xylenes present at 4-7.2 ug/L. Using combinations of sampling, isolation and reversal of water flow, the faulty section of main was identified as YJS. This pipe was isolated and replaced with PVC main. Following main replacement, several samples were analyzed to ensure that the water supply was free of any xylene contamination. The epoxy coating of an unused piece of YJS (similar to that which was removed) was analyzed for volatile hydrocarbons. Scrapings from the surface of the epoxy lining were transferred to a headspace vial and analyzed, using the same method as for the water samples. While not being quantitative, this analysis did show that the same compounds detected in the contaminated samples were present in the coating material (Figure 5). These solvent residues may be present as a result of incomplete hardening of the epoxy or from entrainment of solvent pockets beneath the linings' surface.

The Waterworks Laboratory was involved in investigation of this distribution system incident from December 1989 until March 1990. A total of over 200 samples were analyzed for VOCs and many samples were also analyzed for base-neutral extractable compounds. Cooperation from the affected homeowners was excellent, as was the cooperation between the City and the area developer. The source of the contamination was precisely pinpointed and resolved with no health risk to consumers. If the Waterworks Laboratory had not been able to perform the VOCs analysis, the samples would have be sent to a contract laboratory. This would have resulted in an unacceptably delayed response to the consumers' complaint as well as extremely costly analyses.

FIGURE 5

Chromatograph (dynamic headspace GC-PID) of scrapings taken from the epoxy lining of a YJS pipe contained high concentrations of VOCs and was very similar to chromatograms obtained from water samples from the contaminated YJS main in the distribution system (Figure 4). (A blank chromatograph obtained by analyzing reagent water is shown below the sample chromatograph.)



CONCLUSIONS

Properly installed epoxy-lined steel pipe is a reliable distribution main material, however, contamination of drinking water can occur when the epoxy has not completely hardened before use or pockets of hydrocarbon solvents exist beneath the lining. Despite the contamination of drinking water by YJS pipe reported here, the City of Calgary has obtained good service overall from this water main material and it will continue to be used. This incident has underscored the value of a comprehensive monitoring program for VOCs to collect statistical background data for compliance as well as unusual contamination events.

The method used for VOCs, like for THMs, must be a simple, highly automated method capable of handling large number of samples and running virtually unattended 24 hr per day. The dynamic headspace GC-PID method has proven itself to be a reliable and effective tool for determining volatile hydrocarbons in drinking water. The method provides good sensitivity and accuracy to the level of 0.5 ug/L and compares favorably to P&T sampling. Complete analysis requires only 40 minutes per sample and the use of an autosampler allows virtually unattended operation.

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WATERMAIN REPLACEMENT USING TRENCHLESS TECHNOLOGY
REGINA'S ALBERT STREET UPGRADING PROJECT

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INTRODUCTION

The City of Regina's comprehensive program for renewal and upgrading of its infrastructure includes Albert Street, one of the City's major arteries. In 1989, the West side of Albert Street between Regina Avenue and 25th Avenue was upgraded.

During the 1980s trenchless technology has emerged as an alternative option for the replacement and upgrading of urban infrastructure. The City of Regina took advantage of this emerging technology to replace cast iron watermains as part of the Albert Street Upgrading Project.

REGINA'S INFRASTRUCTURE RENEWAL PROGRAMS

Replacement programs, using Local Improvement procedures were put in place in 1979 with the intention of replacing all cast iron watermains and replacing defective concrete walks, curbs and gutters within a set period. By the late 1980s the programs had evolved and been augmented to form the coordinated program that is in place today.

The objective of the coordinated program is to ensure that when a location is targeted for renewal, all deficiencies are identified and addressed. In addition to watermain and concrete renewal, the scope now includes;

- upgrading and repairing sanitary and storm sewers,
- disconnecting catch basins from the sanitary sewer system,
- installing storm water retention tanks,
- upgrading traffic lights,
- reconstructing deteriorated pavement structures.

Local Improvement Procedures

Watermain replacement and concrete replacement are performed as Local Improvements under the provisions of the Local Improvement Act. Other works are funded and implemented under a variety of capital programs.

Each year, the City proposes a program of Local Improvements and presents it to the abutting property owners. If they wish, the owners can oppose the work by petitioning against it.

Abutting property owners bear a portion of the cost of the improvement. Uniform Rates, equivalent to the cost of new construction in a new development, are set each year. Property owners are assessed Local Improvement Charges based on the Uniform Rates and the extent of their property.

THE ALBERT STREET UPGRADING PROJECT

Albert Street is Regina's main North South artery, extending from Number 1 Highway in the South to the junction of the Number 11 and Number 6 highways North of the City. In terms of traffic volumes, the central portion of Albert Street carries over 33,000 vehicles per day, with peak volumes of over 2,500 vehicles per hour.

In addition to its importance to the transportation system, Albert Street symbolises Regina's claim to the title, "Queen City of the Plains". For 2 km, between College Avenue and 23rd Avenue, Albert Street is flanked to the East by Wascana Centre and the Legislative Building and is lined on the West side by imposing residences set in mature landscaped grounds.

Figure 1 shows the location of Albert Street within the City and identifies the sections where major infrastructure upgrading is planned. The five year Albert Street Upgrading Project covering these locations commenced in 1988.

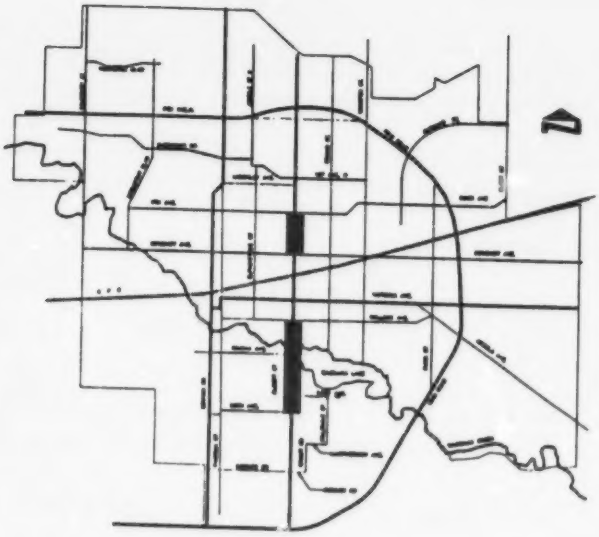
The project includes concrete walks, curbs and gutters, pavement, sewers and watermains, as well as structures, such as the Albert Street Memorial Bridge, aesthetic features, trees, boulevard landscaping, lamp standards, benches, etc.

Prior to implementation, and as a continuing process throughout the project, there has been a high degree of public involvement and participation. As well as the concurrence of abutting owners demanded by the Local Improvement process, there were opportunities for input during a Transportation Impact Study and at a number of public meetings. Special consultative processes were established with residents' committees, heritage organizations and businesses.

The overriding conclusion of the public involvement processes was that the appearance and character of the central portion of Albert Street must be maintained intact. Reconstruction work would have to respect and protect the established trees and streetscape.

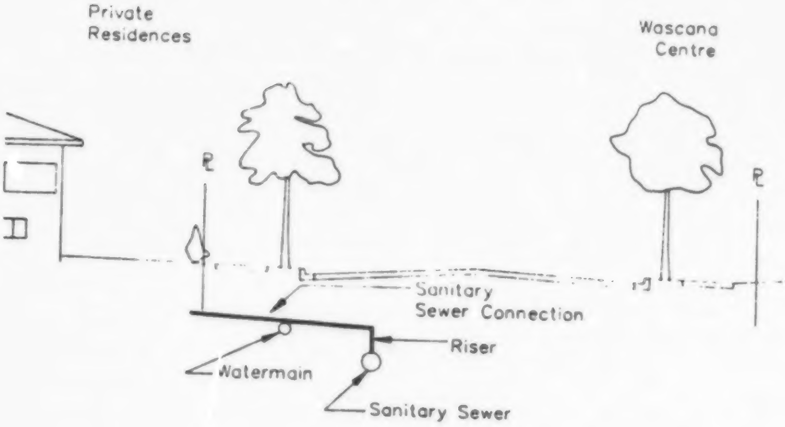
FIGURE 1

REGINA



CITY MAP - SHOWING LOCATIONS OF ALBERT STREET UPGRADING PROJECT

FIGURE 2


Private Residences
Wascana Centre
℄
℄
Sanitary Sewer Connection
Riser
Watermain
Sanitary Sewer

ALBERT STREET, REGINA
TYPICAL CROSS SECTION LOOKING NORTH
SHOWING LOCATION OF UNDERGROUND SERVICES

WATERMAIN REPLACEMENT

Options for Watermain Replacement

Figure 2 is typical a cross section of the central portion of Albert Street, showing the location of the watermain in relation to the walk, pavement and trees. Condition surveys showed the pavement was in good condition where the cast iron watermain was to be replaced.

Selecting a watermain replacement method which satisfied the requirements identified by the public participation and engineering appraisals was not a simple task.

Four Options Were Identified

(1) Install a new watermain under the pavement by trenching or by "coring". (Figure 3)

Advantages:

Clear access for future repair or maintenance.
Use of traditional or proven materials and methods.

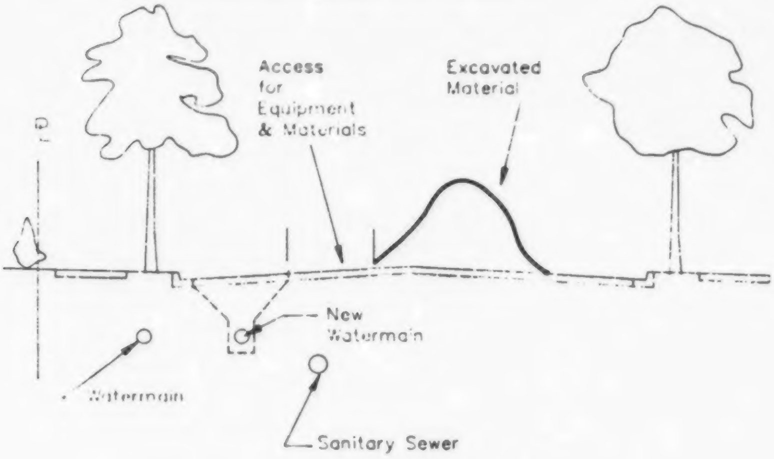
Disadvantages:

High cost of replacing pavement structure.
Long term settlement and poor pavement performance.
Risk of conflict with sanitary sewer connections.
Obstruction and disruption of traffic.
Cored installation may deviate from intended alignment and pipe may be over stressed where deflections are corrected.

FIGURE 3

OPTION 1

NEW WATERMAIN INSTALLED IN TRENCH



(2) Install a new watermain under the walk by trenching or "coring". (Figure 4)

Advantages:

Less disruption to traffic.
Eliminates pavement cuts and settlement.
Uses familiar pipe material.

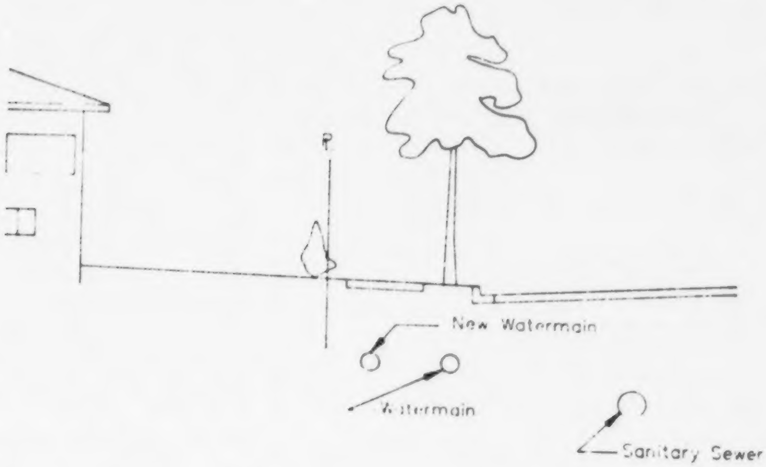
Disadvantages:

High risk of damage to trees and landscaping.
Confined working area.
Risk of conflict with sanitary sewer connections.
Trench settlement would damage concrete walk.
Deviations and stress in pipe if installed by coring.

FIGURE 4

OPTION 2

NEW WATERMAIN INSTALLED UNDER SIDEWALK



(3) Replace the watermain in its existing location using a trenchless technique.

Advantages:

Minimum disruption to traffic.
Least risk of damage to trees.
Eliminates pavement cuts and settlement.

Disadvantages:

Temporary water supply needed during construction.
Involves unfamiliar materials and techniques.

Materials and equipment for repair not immediately available.
Limited number of specialist contractors.

(4) Do not replace the watermain.

This option was rejected at an early stage. Although the watermain had a relatively good record of reliability, it was by no means trouble free. Constructing new surface works over a potentially unreliable watermain was not an acceptable option.

Selection Criteria

Criteria for selecting a method for replacement were established. These included:

Eliminate, as far as possible, long term pavement settlement.

Minimise extent and cost of pavement replacement.

Protect the established trees and landscaping, both public and private.

Minimise disruption to traffic movement.

Ensure compatibility between any new materials and current maintenance and operational practices.

Each of the options was investigated. Contacts were made with specialist contractors where new methods or technologies were involved. Costs were estimated and the impact of each option on the system operators was considered.

TRENCHLESS REPLACEMENT

After the initial investigation it was concluded that replacement of the watermain in the same location using a trenchless technique was viable and offered advantages. This option was then studied in detail to identify and solve a number of potential problems.

Problems can be grouped under three headings. Namely, those problems related to the method of construction, the material used for the replacement watermain and the contractual arrangements for construction.

Method of Replacement

When the search for a method of replacement started early in 1988 there was almost no industry experience of "in place" cast iron watermain replacement. A number of North American companies were in the process of developing equipment and techniques. The main thrust to develop trenchless replacement techniques had come from Europe and contacts were made with the North American agent for the British PIM method. While several versions of this type of method now exist or are being developed, at the time the PIM Corporation was the only North American source of a method with demonstrated capability and the back up of an experienced organization.

While it is not the purpose of this paper to describe trenchless methods in detail, a brief overview will assist the understanding of the steps involved.

The Pipe Insertion Method

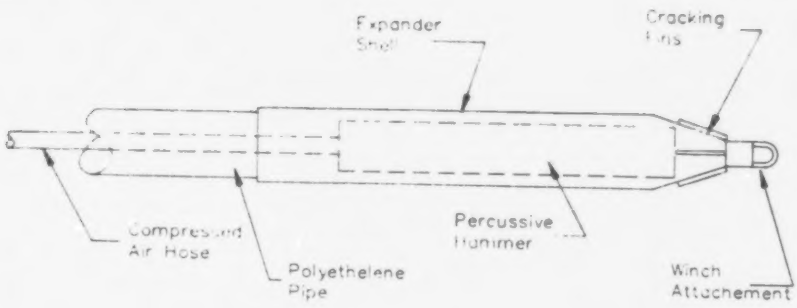
The Pipe Insertion Method (PIM) is a method of inserting a new pipe into an existing pipe by pipe bursting. A pneumatic or hydraulically powered tool breaks the pipe and expands it, forcing the fragments outwards into the surrounding earth.

Originally conceived in the United Kingdom as a method for replacing deteriorated natural gas mains, the method was developed jointly by the British water and gas industries. The PIM method is now protected by patents held by British Gas Plc. Since 1983, the method has been developed for the North American Market by The PIM Corporation of Piscataway, New Jersey.

Figure 5 shows the pneumatic pipe bursting tool and figure 6 shows the arrangement of the main pieces of equipment.

FIGURE 5

PIM INSERTION TOOL



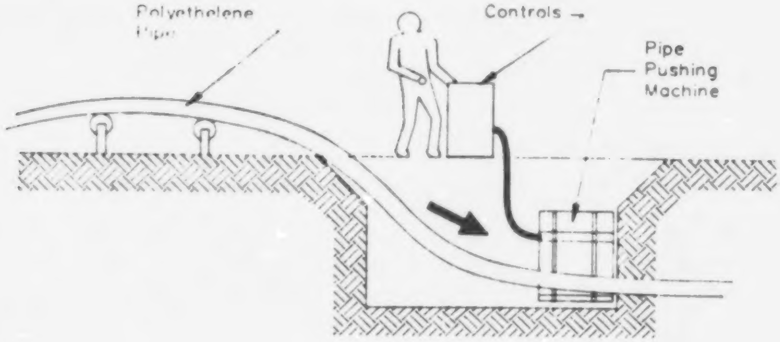
The tool contains a pneumatic hammer which provides the power to break the pipe and expand the hole. A hose, threaded through the new pipe, supplies compressed air to the tool. A cable connects the nose of the tool to a constant tension winch. The winch does not contribute much force to the bursting operation but holds the tool in continuous contact with the pipe, ensuring that the full force of the pneumatic hammer is brought to bear.

As the tool moves through the pipe, polyethylene pipe is pushed into place behind it. The hydraulically powered pushing machine has an arrangement of clamps and cylinders which grip the pipe and move it forward, keeping pace with the tool.

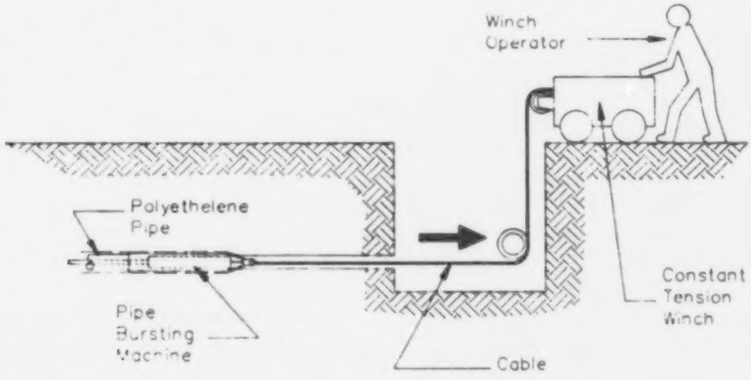
For "non pressure" applications such as concrete or VCT sewers and some "pressure" applications the PIM process achieves replacement in one step. For Cast Iron Watermain replacement a sleeve pipe is inserted first, then the new watermain is installed inside the sleeve pipe. This arrangement is necessary to ensure the new watermain is not cut by fragments of cast iron.

FIGURE 6

ARRANGEMENT AT LAUNCH PIT


Polyethelene Pipe
Controls →
Pipe Pushing Machine

ARRANGEMENT AT RECEIVING PIT


Winch Operator
Polyethelene Pipe
Constant Tension Winch
Pipe Bursting Machine
Cable

Installation by Coring

At the same time that the PIM method was being investigated, care was taken to make sure alternative options were kept open. The most appropriate alternative method was considered to be to use earth coring to install the watermain under the walk. This would minimise excavation volume, disruption and settlement but had the disadvantages listed previously.

In the coring process a hole is formed between two pits. First a rod is driven through the earth at the desired grade and line. A variety of tools is used to expand the hole to the required diameter and special tools and techniques are available to correct minor deviations from the intended line.

The City has used coring many times to install underground utilities under obstructions or in congested locations. Regina's uniform clay soil gives the method a good success rate but disturbed ground, hard dry ground or obstructions can cause deviations. Range is limited and the method has not been used extensively over the distances needed for the Albert Street watermain replacement.

Construction Details

The choice of "in place" trenchless replacement meant that the new pipe would be polyethylene. This decision gave rise to problems which can be classed into two groups, those related to the material's properties and those related to the need to ensure that the City's crews would be able to maintain and modify the watermain in the future. Some of the methods that were used to deal with this aspect are dealt with below.

Pipe Joints

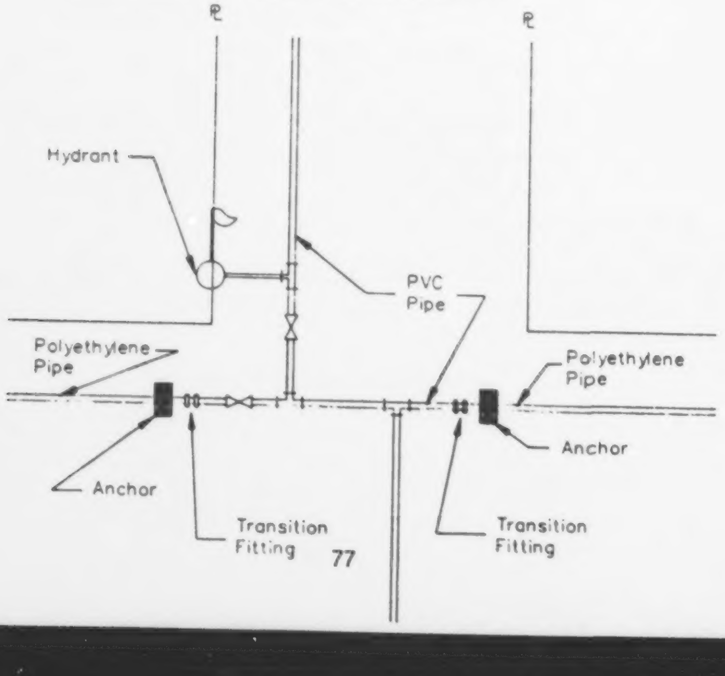
The PIM method requires a continuous length of pipe for the sleeve and replacement pipes. Polyethylene pipe, being readily joined by the butt fusion process is uniquely suitable for the purpose. However, while butt fusion provides a reliable method for joining the lengths of carrier pipe, polyethylene is not directly compatible with the standard PVC pipe and fittings used by the City of Regina.

At each intersection the watermain may connect to a main on the adjoining avenue. There are also valves, hydrant leads etc. which must be incorporated into the system. In the future valves will have to be replaced and there is always a possibility that modifications or additions will have to be made.

It was decided to use PVC pipe in the intersections to give the greatest freedom for future changes and the simplest approach for repairs. Figure 7 shows details of the typical arrangement at intersections.

FIGURE 7

ARRANGEMENT OF PIPEWORK AT INTERSECTION



Thermal Properties

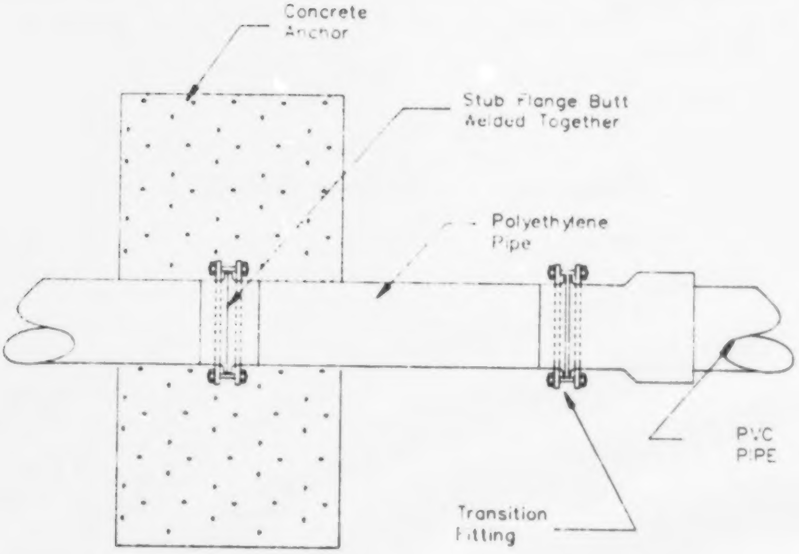
Polyethylene, like most of the plastics used as pipe materials, has a high coefficient of thermal expansion. While this fact is well known to those who use polyethylene pipe and construction techniques have been devised to overcome it, the installation on Albert Street posed particular problems.

Regina's water has a relatively large seasonal temperature range. Temperatures in the distribution system can vary from a low of 2 or 3 deg C in winter to a high of over 20 deg C in summer. Such a variation would, in an unrestrained one block length of pipe, result in a change in length of 400mm.

In the PIM method, installation of the replacement water main within a sleeve pipe means that the pipe is completely unrestrained. Hence measures were needed to restrain the pipe and overcome the seasonal temperature effects. Even when the ends of the pipe are fully restrained there is still a possibility that small lateral movements could occur as the pipe goes from a state of tension to a state of compression each spring. Figure 8 shows the arrangement that was used to anchor the ends of the carrier pipe.

FIGURE 8

ANCHORING/TRANSITION DETAILS



Plastic Behaviour

By definition plastic materials deform permanently under load. The rate of deformation is controlled by the properties of the plastic and the intensity of the load. Polyethylene is known to deform readily, being more plastic than PVC. For this reason the normal construction techniques may not be suitable for polyethylene. This is particularly true of the method of connecting services.

A method of making service connections was needed which did not apply high stresses to the pipe. Consideration was given to a number of methods for overcoming this problem. The method chosen was to use a repair clamp that would subject the pipe wall to a low pressure over a relatively large area.

Enquiries showed that this type of clamp had been used successfully. The method was chosen because it has the advantage that repairs or alterations can be made using normal hand tools.

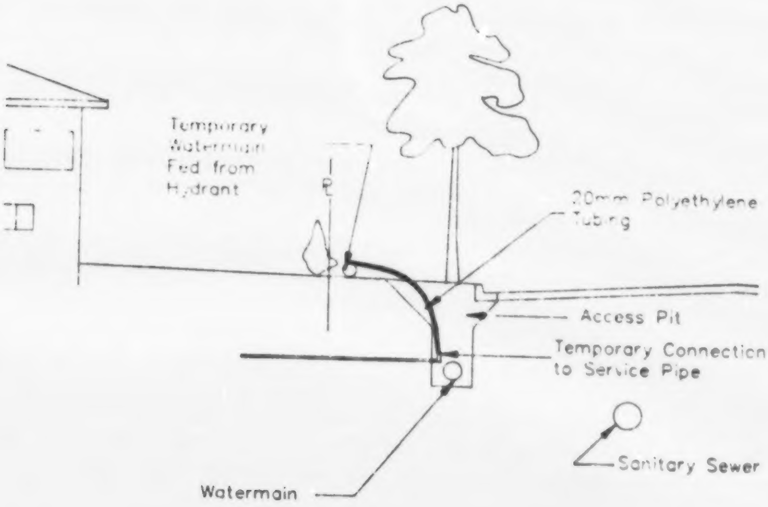
An alternative approach which was also explored was to use fusion welded fittings. This method would eliminate the problem of deformation under pressure but the technique is not yet common for pipes of the sizes used and no local experience was available. Also, there is no known non destructive method for checking the quality of this type of welded joint to ensure complete bonding. In view of the lack of local resources and uncertainties and the system operators' concern that a method of repair must be readily available, mechanical fittings were chosen. [See Footnote.]

Temporary Water Supply

In conventional watermain replacement, water service is interrupted during the day and restored each night by reconnecting the new main to the old. Residents suffer minimal disruption and special provision for continuous service is made only for commercial premises where the loss of water service would have a damaging effect on the business.

FIGURE 9

TEMPORARY WATER SUPPLY



"In place" replacement requires that the watermain to be out of service for several days at a time. An alternative water supply is needed and for the Albert Street project this was provided by a temporary watermain laid on the surface and fed from a hydrant. As each service connection was excavated and disconnected from the Cast Iron main, it was reconnected to the temporary main. Maintaining the temporary water supply was a complicating factor when planning the work and selecting locations for hydrants and valves. Figure 9 shows the arrangement that was used.

TENDERING AND CONTRACTURAL ARRANGEMENTS

Consideration of contract arrangements centred on making the choice of Contractor and specialist subcontractor compatible with the City s purchasing policies. Eventually this problem was solved by an arrangement in which general contractors submitted competitive bids including the services of a specialist subcontractor or a licensing arrangement for the trenchless technology. The City maintained the right to accept or reject the tenderer's chosen method.

Tenders were received from a number of Local contractors. As well as a conforming bid using "in place" replacement, the low bidder also submitted an alternative offer tc install the watermain under the walk by coring. Serious consideration was given to this proposal and in discussion with the luw bidder, a number of proposals using a combination of coring and various trenchless "in place" replacement methods were costed out.

Eventually a hybrid proposal was adopted in which the coring method was used for the Southern part of the work where the distance between house services, and hence access pits, was short, there were no trees and access was good. The PIM method was used for the long blocks where access was more restricted and the distance between connections exceeded the distance that could be accurately achieved by coring.

In the final arrangement 480 metres of PVC watermain were installed by coring and 1,300 metres of polyethylene watermain were installed by the PIM method.

CONSTRUCTION EXPERIENCES

Installation by Coring

Construction proceeded with few problems and the expected rates of progress were maintained. The 480 metres of watermain, covering three and a half short blocks was installed in a period of one and a half weeks. In practice it proved more difficult than expected to minimise the size of the excavations and hence the amount of dirt stockpiled on the street. This, coupled with the need to continually move equipment into place resulted in more traffic disruption than was expected.

Backfilling operations also proved to be more disrupting and extensive than had been planned. It had originally been expected that the size of the core pits would be minimised and that compacted granular backfill would be used The method was changed to permit backfilling with native clay material compacted with a vibrating sheepsfoot roller.

Installation by the PIM Method

PIM installation was performed on seven blocks covering a distance of 1,300 metres. For five blocks the 200 mm diameter watermain was replaced by a watermain of equal size. For the two most Northern blocks the replacement main was upsized from 200 mm to 250 mm.

The Contractor's schedule anticipated a rate of progress of one block of replacement each week. By and large this rate was maintained once some minor equipment problems and scheduling difficulties were overcome. The various activities involved can be broken down into three stages, preparatory work, replacement and completion.

Preparatory Work

Space does not permit a detailed description of the preparatory steps, however the main tasks were:

- Locate and excavate service connections
- Install temporary water supply and connect service connections
- Uncover and prepare sanitary sewer connections
- String and weld sleeve and carrier pipes
- Install winch and pipe pushing machine.
- Prepare PIM tool and compressed air system.

Generally the preparatory tasks took about a week to complete.

Replacement

Replacement of the watermain itself was, with one exception completed in one working day. The exception was for one block where the drive was halted after 11 hours and resumed the next day. The slow progress was due to problems with the compressed air system, combined with very hard ground conditions. By contrast the quickest installation took 8 hours. It is interesting to note that the quickest installation was on one of the two blocks where the watermain was upsized. The faster rate was achieved because a larger, more powerful tool was used.

After the sleeve pipe had been inserted, the ends were cut to length and sections cut out at each service connection. The carrier pipe was then slid into place and left to rest overnight. The rest period allows the pipe to cool to ground temperature and relax, releasing installation stresses. The next day completion of the installation commenced.

Installation by the PIM method proceeded very much in accordance with expectations and there were very few problems with the watermain replacement itself.

During the drive, sewer service connections which were close to the watermain had to be disconnected and, once the tool had passed by, reconnected. Dealing with the sanitary sewer connections and the arrangements for reinstating the service connection access pits was the biggest source of unforseen difficulties during the work.

Difficulties arose because it was not possible to predict the proximity of the sewer connections and the extent of the measures needed to protect and reinstate them prior to installation. Provisions in the Contract document and instructions at the tender stage were intended to make these aspects clear. However, the City's interpretation was not shared by the Contractor. Resolution of this problem resulted in extra cost.

Completion

The main tasks needed to complete the watermain replacement were;

- Fitting anchor flanges and casting thrust blocks.
- Fitting service connection clamps
- Connecting to the distribution system.
- Pressurising, chlorinating and flushing.
- Attaching service connections.
- Rechecking for watertightness.
- Backfilling.
- Surface restoration.

Completion of the installation usually took a further week.

COSTS

In 1988, total cost for watermain replacement was estimated to be:-

Replacement of watermain		$450,000
Pavement reinstatement		$380,000
	Total	$830,000

The final total cost was $570,433 of which $507,896 was for services provided by the contractor and the balance was the City's costs for engineering, administration, and other City supplied services.

The cost of the two trenchless installation methods have been calculated. The figures below include the cost of the pipe material, (including the sleeve pipe for the PIM method), the connection clamps and the cost of installation. No allowance is included for work in intersections, valves, hydrants, etc., surface reinstatement or for any administrative or engineering costs.

PVC watermain by coring,	$190 per m
Polyethylene by PIM method	$232 per m

It is difficult to make a generally applicable conclusion about the relative costs and benefits of the two trenchless methods, or for that matter the cost of trenchless installation compared to replacement by trenching. Clearly, the final cost was significantly less than the original estimate. However direct comparison to the estimate might not be valid because the estimate may not be the true cost for comparable work.

EVALUATION

When measured in terms of its cost and performance objectives, the project was a success. Disruption of traffic, damage to trees and underground facilities and long term settlement were all minimised. Many residents took a particular interest in the job and were very positive about the benefits which they perceived.

Problems with the equipment or the technique were rare and were solved by the operators as the work progressed.

It has to be said that the City's operations staff have concerns about the installation that they will inherit at the end of the Contractor's warranty period. Polyethylene is a new material in Regina's water distribution system and requires a new inventory of equipment and new techniques for maintenance or repair.

Operator's concerns must always be a part of the consideration and selection of a trenchless replacement method, as they were in Regina. However, having identified the concerns and given them proper consideration the final choice must be based on all the relevant factors.

The experience gained on the project was very positive and the City will certainly consider trenchless techniques for future work. The project also showed the importance of thorough planning and attention to detail.

In closing, the authors acknowledge the roles played by Wappel Construction Ltd., Mercury Earth Coring Ltd., The PIM Corporation and other City Departments in the successful completion of the project. It should be noted that opinions contained in the paper are those of the authors and are not necessarily those of the City of Regina.

FOOTNOTE

At the time of writing, (June 1990), the City is dealing with a problem that has arisen with some of the water service connections. Full details and the extent of the problem are not yet known but it is believed to involve the service clamps. The authors suggest that anyone intending to use this type of connection should contact the City of Regina for updated information.

USE OF INEXPENSIVE ADSORBENTS FOR THE REMOVAL OF CERTAIN PRIORITY POLLUTANTS FROM WASTEWATERS

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INTRODUCTION

Contamination of groundwater from human activities may come from surface impoundments, landfills, agriculture, leaks and spills, land disposal of wastewaters and hazardous wastes, septic tank systems, mining, petroleum and natural gas production, underground injection wells, and other sources (1). Heavy metals such as cadmium, chromium, copper, nickel and zinc are present generally at significant concentrations in municipal wastewater (2,3). The monitoring data from both federal and state studies in U.S.A. on affected groundwater supplies showed that of all the groundwater supplies collected and analyzed, trichlorethylene (TCE), an industrial solvent and degreaser, was detected more frequently and in the highest concentration and that tetrachlorethylene ranked second in frequency of occurrence (4). According to a recent U.S. EPA report (5), the toxic compounds listed below represent the likely most frequently occurring organic compounds in domestic wastewater.

1. benzene
2. phenol
3. 2,4,6-trichlorophenol
4. 2-chlorophenol
5. 1,2-dichlorobenzene
6. 1,4-dichlorobenzene
7. 1,1,1-trichloroethane
8. naphthalene
9. toluene
10. diethylphthalate
11. dimethylphthalate
12. trichloroethylene
13. aldrin
14. dieldrin

Most of these compounds are contributed by household cleaning products and cosmetics. In the case of municipal wastewaters, trace organics could be contributed from certain industries discharging into a municipal sewer system. Many of these trace organics have been identified in municipal wastewaters (6,7,8,9) and in septic tank effluents (10, 11, 12). A large number of villages and towns in Canada use stabilization ponds and aerated lagoons for wastewater treatment, with a potential for groundwater contamination involving heavy metals and trace organics (13).

Studies by Coupal and Lalancette (14) and Cameron (15) showed good removals of heavy metals using peat; reduction in 2,4-D has been reported using peat (16). However there is limited information on the removal of heavy metals and trace organics from municipal wastewaters using peat.

Liners (synthetic or natural) have been used in waste treatment lagoons and in hazardous waste landfills essentially to minimize the seepage reaching the groundwater and contaminating the aquifer. Bentonite and soil-bentonite liners have been extensively used for seepage control (17,18); however, information on their capability for contaminant attenuation is very limited especially in respect of trace organics and heavy metals. Bentonite is a naturally occurring clay consisting of 70 percent to 90 percent of the clay mineral montmorillonite. Bentonite mined and processed in Avonlea in southern Saskatchewan contains about 80 percent montmorillonite, 7 to 10 percent illite and 5 to 8 percent quartz. Sodium bentonite, the kind available in Avonlea, which is most often used in civil engineering applications, swells up to 15 times its dry bulk volume when exposed to water. Avonlea bentonite has a high specific surface area and a high cation exchange capacity (CEC) (19). These properties of the bentonite make it a potential medium for adsorption of metals and organics. Although some work has been carried out on the removal of heavy metals by clay and clay-sand mixtures, studies related to the removal of trace organics or heavy metals by bentonites are limited. The effect of some organic solvents on permeability of certain clay soils has also been studied earlier.

INEXPENSIVE ADSORBENTS AND REMOVAL OF HEAVY METALS

In spite of the effectiveness of activated carbon in the removal of heavy metals from wastewaters, the high cost of activated carbon has restricted its more widespread use. Since adsorption is one of the more effective methods for removing heavy metals from wastewaters, an inexpensive and more easily available adsorbent would make the removal of heavy metals from wastewaters by adsorption an economically viable alternative. It has been demonstrated that inexpensive adsorbents such as tree bark, discarded automotive tyres, human hair, sawdust, corncobs, peanut hulls, straw, coffee grounds, coconut shells, and rice husks are effective in removing heavy metals from wastewaters (20,21,22,23,24,25,26,27,28,29,30). The abundance of peat and fly ash and their easy availability make them economical adsorbents for removing heavy metals from wastewaters.

Batch studies were therefore conducted to evaluate the possible use of locally available horticultural peat and fly ash in the removal of cadmium and chromium from municipal wastewater.

USE OF PEAT AND FLY ASH FOR HEAVY METAL REMOVAL

PEAT

Peat can be described as fossilized plant matter which occurs in wet areas where there is a lack of oxygen; the accumulation of plant metteris therefore more rapid than its decomposition. Peat moss is a rather complex material containing lignin and cellulose as major constituents. These constituents, especially lignin, contain polar functional groups, such as alcohols, aldehydes, ketones, acids, phenolic hydroxides and ethers that can be involved in chemical bonding (14). Because of the very polar nature of this material, the specific adsorption for dissolved solids such as transition metals and polar organic molecules is reported to be quite high.

Peat deposits varying in thickness from a few metres to tens of metres are found throughout the world. Besides being plentiful and inexpensive, peat has several other characteristics that make it an effective medium for wastwater treatment. Peat has a greater cation exchange capacity than mineral soil and research by several investigators has show that peat is an effective adsorbent for the removal of heavy metals from wastewaters (14,31,32,33,34,35).

FLY ASH

Fly ash is defined as the finely divided residue that results from the combustion of ground coal. Fly ash is a particulate substance ranging from 0.5 to 100 microns in size and composed primarily of silica (SiO_2) and alumina (Al_2O_3) with variable amounts of iron oxides (Fe_2O_3). Other macroelements present to a greater or lesser degree include carbon, calcium (as lime or gypsum), magnesium, sulfur (as SO_4^{-2} and SO_3^{-2}), sodium and potassium (36).

The use of fly ash in wastewater treatment has been studied in recent years and experimental results have shown fly ash to be a suitable adsorbent for the removal of heavy metals (cadmium and chromium from wastewaters (37,38,39).

MATERIALS AND METHODS

MATERIALS

The wastewater used in the study was collected from the City of Regina Municipal Wastewater Treatment Plant, before the wastewater enters the primary sedimentation tanks. The wastewater was spiked with stock solutions of cadmium and chromium to raise their concentrations to approximately 1 mg/L.

The sphagnum peat moss used in this study was horticultural peat manufactured by Premier Peat Ltd., Carrot River, Saskatchewan. The following two types of peat were studied for their adsorption capacities:

1. sphagnum peat moss sieved by #20 mesh and dried at 103°C for 24 hours; and
2. sphagnum peat moss sieved by #20 mesh, shaken thoroughly with 1 M HCL for 2 hours, washed with deionized water until the filtrate reached a pH of 4, and dried at 103°C for 24 hours.

The fly ash used in this study was obtained from unit #2 of the Poplar River Power Station of the Saskatchewan Power Corporation. The fly ash generated from the combustion of lignite was collected from the first bunk of the power station's electrostatic precipitators.

METHODS

The batch (kinetic and adsorption isotherm) studies were conducted using a Jar Test Apparatus. Batch adsorption studies were conducted at various pH values and the best pH for maximum adsorption was chosen. Isotherm studies were conducted at various temperatures (5°C, 10°C, 15°C and 21°C). Desorption studies were conducted at 21°C. Methods outlined in "Standard Methods" (40) were followed for the analyses.

SUMMARY RESULTS OF BATCH STUDIES (41)

Results of batch studies conducted showed that peat was effective in removing 95% of cadmium and 53% of chromium in the wastewater at 21°C in the pH range 4 to 5 in an equilibrium time of 2h. The adsorption of cadmium on peat was exothermic, while the adsorption of chromium on peat was endothermic. Kinetic studies indicated the reaction rate constants for cadmium and chromium adsorption on peat to be 0.0134 $hour^{-1}$ and 0.0379 $hour^{-1}$ respectively. Acid-treated peat was found to adsorb 80% of the cadmium and 55% of the chromium in the wastewater at 21°C degrees centigrade in the pH range 4 to 5 in an equilibrium time of 3 h. The reaction rate constants for the adsorption of cadmium and chromium on acid-treated peat were found to be 0.0286 $hour^{-1}$ and 0.0493 $hour^{-1}$ respectively. Isotherm analysis of the data showed that the adsorption pattern for cadmium on peat followed the Landmuir isotherm at 5°C, 15°C, and 21°C, and the Fruendlich isotherm at 10°C, whereas the adsorption pattern for chromium on peat followed the Langmuir isotherm at 5°C, 10°C, and 15°C, and the Freundlich isotherm at 21°C. The adsorption of cadmium and chromium on acid-treated peat at 21°C was described by the Freundlich and Landmuir isotherms respectively. The acid-treatment of peat improved the cadmium and chromium adsorption capacity of peat.

Batch studies showed that fly ash removed 93% of the cadmium in the wastewater in the pH range 7 to 8, and 44% of the chromium in the wastewater in the pH range 2 to 3 in an equilibrium time of 3h. The adsorption of cadmium and chromium on fly ash was found to be exothermic. Isotherm analysis of the data showed that the adsorption of cadmium on fly ash was described by the Langmuir isotherm at 5°C, 10°C, 15°C and 21°C, whereas the adsorption of the chromium on fly ash followed the Freundlich isotherm at 5°C, 10°C, and 15°C, and the Langmuir isotherm at 21°C. The Langmuir and Freundlich isotherms showed that fly ash had a higher adsorption capacity for cadmium compared to peat and acid-treated peat, whereas acid-treated peat had a higher adsorption capacity for chromium compared to peat and fly ash.

APPLICATION TO PRACTICE

The results of this study can be used to design batch adsorption systems for the treatment of wastewaters contaminated with cadmium and chromium. Batch adsorption studies showed that peat effectively adsorbed 95% of the cadmium and 53% the chromium from wastewater, while fly ash adsorbed 93% of the cadmium and 44% of the chromium wastewater.

For systems where the total volume of wastewater is small and generated over a period of 8 to 12 hours, one reaction vessel may be adequate for the treatment of the wastewater. In cases where a continuous system is required, two reaction vessels may be used alternatively using each of the reaction vessel for the treatment of the wastewater. Peat or fly ash can be added to the reaction vessel containing the wastewater and the mixture may be agitated for 2 to 3 hours (equilibrium time) such that the maximum amount of cadmium and chromium are adsorbed on to peat or fly ash. The wastewater and fly ash can then be separated after agitation to prepare the reaction vessel for another batch of the wastewater. Since the Langmuir isotherm modelled the adsorption of cadmium and chromium on peat and fly ash, the amount of peat and fly ash required to decrease the concentration of cadmium and chromium in the wastewater can be calculated. The design capacity of the reaction vessel will reflect the minimum reaction time (equilibrium time) required and the volume of the wastewater to be treated.

The used peat can be drained, dried and used as a fuel or disposed of in a landfill. The used peat as a fuel may require careful consideration because the disposal of the metal-enriched ash after the combustion of peat would pose further disposal problems. The used peat can be disposed of in a landfill with minimum environmental effects or used as a filler material for pavements. Fly ash can be disposed of for use in the cement and concrete industry, or can be used as a fill material for roads and construction sites and as a filler in asphalt mix.

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FILTRATION & PRETREATMENT - AN OVERVIEW

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GENERAL

Filtration, as applied to water treatment, is the process of separating suspended or colloidal solids from water by passing it through a porous medium. Two basic types of filter are in common use: pre-coat (diatomaceous earth) filters and granular filters. The former use a very thin layer of extremely fine (less than 100 micrometers) filter medium which is wasted after each filtering cycle. The latter use relatively coarse (typically 0.35 - 1.5 mm) filtering media such as sand, coal, garnet and ilmenite which is cleaned in place after each filtering cycle. Cleaning is accomplished in a variety of ways and is discussed later. This paper confines itself to a discussion of granular filters commonly employed in municipal water treatment.

HISTORY OF FILTER DEVELOPMENT

Slow sand filters were first used on a wide scale for public water supplies in England in the early 1800's. They employed about 1m of ungraded fine sand over a gravel bed in which was placed a perforated pipe collection system. These early filters were operated at between 0.015 and 0.033 m/h on raw untreated water. They were taken out of service for cleaning about every six months. Cleaning was accomplished by scraping off the top 75 - 100 mm. The prime purpose of the filters was the removal of particulate matter from suspension in surface water; not the protection of the public health. They were employed on raw water and provided the only treatment used.

In 1854 an outbreak of cholera in London claimed over 500 lives. The source of the disease was traced to a water well which had been contaminated by sewage from a broken drain. Further investigation showed that a soldier, recently returned from service in India, was the carrier of the disease and was a resident of the rooming house served by the broken sewer. About the same time, Pasteur and others were formulating the germ theory of disease. This, and the knowledge that disease could be transmitted through the medium of drinking water, led to authorities adopting filtration of water supplies as a measure of protecting public health. It was soon realized that slow sand filters were biologically active systems capable of removing pathogens and other organisms from raw water. The primary role of the slow sand filter gradually changed from that of removing particulates from the water to that of protecting the public health by removing pathogens.

The Hamburg cholera epidemic in the 1890's provided convincing proof of the effectiveness of slow sand filters in this role. By the end of the 19th century, slow sand filters were widely used throughout Europe.

Slow sand filters did not enjoy the same popularity that they had in Europe when they were introduced by North America. This was due mainly to the very short filter runs occasioned by the clay bearing waters encountered on this continent. They were also labour intensive at a time when industry was beginning to attract labour with higher wages. Rapid sand filters preceded by chemical treatment, flocculation and sedimentation, were developed as an alternative. They were typically operated at between 4.5 - 6.0 m/h (200 - 250 times the rate of slow sand filters) at an operating head of 3 - 3.7m. George W. Fuller's work, using the Ohio river water at Louisville, Kentucky at the turn of the century, set the stage for the development of the American, or rapid sand, filter. The design criteria he developed remained the guidelines for practically all potable water filters built for the next fifty years.

Again, the prime purpose of the filter was protection of the treated water against pathogenic contamination. Chemical pretreatment was intended to reduce the load going on to the filter and allow it to be operated effectively. Rapid sand filters employed 0.75 - 0.9m of graded sand with an effective size of abour 0.45mm and a uniformity coefficient of 1.6 - 1.75. The filtering cycle was terminated when the head loss across the filter reached about 2.5m; generally after one to two days. Cleaning of the filters was accomplished by reversing the flow at a rate high enough to fluidize the filter medium, in the order of 3 m/h, and discharging the wash water to waste. In this way filters could be cleaned frequently and with the minimum amount of labour.

In the early part of 20th century chlorination was developed as a means of disinfecting water supplies and soon became accepted as the major factor in protecting the consumer. The primary role of the filter reverted to that of particulate removal while it remained as a secondary barrier against disease.

Backwashing filters had the effect of stratifying the filter medium, leaving the finer sand grains in the upper part of the filter and the larger grains at the bottom. Consequently most of the filtering action took place in the top layers of the filter. Dual media filters were developed in the late 1950's and early 1960's to allow greater depth penetration of particulates and higher operating rates while at the same time producing a better quality effluent. Dual media filters employing 450 mm of anthracite overlying 300 mm of silica sand have been installed in major water treatment plants in Canada and the United States since 1972. Effective sizes are typically 1.1 mm for anthracite and 0.45 mm for sand. Operating rates of 12 - 15 m/h are common.

Mixed media filters (also known as tri-media filters) comprising anthracite, silica sand and garnet in descending order from the top of the filter were introduced about the same time as the dual media filters. Ilmenite is often used in place of garnet. The anthracite and sand are generally similar in physical properties to that employed in dual media filters. The garnet is a very find dense medium having an effective size of 0.15mm and a specific gravity of 4.2. Mixed media filters were originally designed to provide some intermixing of the media at the interface between separate layers; hence the term "mixed media". In some cases, the media is designed to avoid such mixing and a very sharp interface is maintained between layers. Both mixed media and dual media filters retain their initial stratification following backwashing because of the different specific gravities of the various media; the finer media having higher specific gravities than the coarser media.

Although mixed media filters have been successfully employed in many water treatment plants, both municipal and industrial, dual media filters have been more readily accepted because of their lower cost and greater simplicity. Mixed media and dual media filters appear to perform equally when chemical pretreatment of the water is optimal and especially when a polymer is used as a filter aid.

Deep bed monomedium filters using coal, sand or other materials have recently been receiving attention. Particle deposition occurs throughout the coarse monomedium filter leading to higher storage than with conventional filters. Backwashing must be supplemented by air scour.

The Los Angeles Aqueduct Filtration Plant employs monomedium filters, using 2 metres of 1.5mm effective size anthracite over a thin gravel base. The filters are operated at up to 33 m/h, or about three times the rate commonly employed with dual media filters. The plant uses ozone as a preoxidant and to promote microflocculation with ferric chloride and cationic polymer prior to direct filtration. Filter run lengths in excess of 30 hours are reported.

The Happy Valley Water Filtration Plant serving Metropolitan Adelaide, Australia includes deep bed coarse sand filters designed to be operated at 20.7 m/h with a minimum run length of six hours. The filters, which follow conventional mixing, flocculation and sedimentation, consist of 1.5 m of 1.5 mm size sand.

Deep bed coarse monomedium filters using indigenous materials other than sand and anthracite have been used in India, Brazil, Chile, Colombia and Philippines.

A greater understanding of the physical and chemical processes involved in destabilization of colloidal dispersions, together with improvements in rapid mixing and flocculation, has led to higher quality water going on to the rapid granular filter. In fact, by the 1960's it was generally accepted that pretreated water should have a maximum turbidity of about 5 NTU prior to filtration; a value widely used in drinking water guidelines for filtered water. Our own [Associated Engineering] standard specifications for upflow clarifiers required a guaranteed performance of a maximum turbidity of 5 NTU. The filter's role in particulate removal is now equally shared by chemical pretreatment.

The primary role of granular filters has shifted back and forth between that of particulate removal and of protection against disease. It is true to say that with modern day knowledge of chemical treatment and filtering mechanisms the rapid granular filter is well established as an integral part of the water treatment process, helping in the removal of particulate matter from the water and providing one of the multi barriers against disease. It should not be regarded as being separate from, not can it be truly effective without, preoxidation, chemical pretreatment, mixing, coagulation and sedimentation and disinfection.

SLOW SAND FILTERS IN THE TWENTIETH CENTURY

Low turbidity water treated by chlorination only has long been used for public water supplies in many parts of Canada and the U.S. Vancouver, Whitehorse, Seattle and San Francisco are a few examples. In recent years the increasing incidence of giardiasis has resulted in a growing concern over the use of untreated water. This is particularly so because the Giardia Lamblia cyst appears to flourish in cold clear waters such as those found in many of our mountain areas. The outbreak of giardiasis in Banff a few years ago demonstrates the vulnerability of our "pristine waters" to contamination of this nature.

Outside of the mountain areas many communities draw their water supplies from sources impounded for irrigation purposes. These water sources are very often low in turbidity and colour. However, they usually have a somewhat higher biological or nutrient load than do the mountain lakes.

Slow sand filtration may be a viable option for those smaller communities facing the problem of constructing treatment facilities for low turbidity source water. The size of filters required and high labour intensity of the filter cleaning process would seem to preclude their use for larger communities. The city of Denver, however, still treats part of its water supply using slow sand filters.

Recent amendments to the Safe Drinking Water Act in the U.S. and the growing realization that even the cleanest water can be easily contaminated have caused a renewed interest in slow sand filters. Research into them has been more limited than into rapid granular filters but certain process variables have been identified as influencing filtration efficiency. These include schmutzdecke development, filter aging, temperature, sand size, filter depth and hydraulic loading.

As stated earlier, slow sand filters are biological systems for treating water; they are not merely granular strainers. The zoogleal mat or schmutzdecke, that forms on top of the filter is able to break down much of the organic matter in the water. It will also entrap and remove colloidal clay particles from the water, leading to a reduction in effluent turbidity. Biological activity within the filter itself continues the process. Exocellular polymers produced by microorganisms within the filter bed will assist in turbidity removal.

Biological activity plays an important part in removal of Giardia and total coliform bacteria. Removal of these organisms is considerably reduced after scraping off the schmutzdecke and even more adversely affected when the sand is replaced entirely. These effects support the view that biological activity is crucial to an efficiently operating slow sand filter.

Bacteria removal is generally regarded as being lower with decreasing temperature. Slow sand filters are, however, used effectively on mountain water sources in temperate and cold climates.

Slow sand filters usually employ a lower grain sized sand than that used in rapid filters. Consequently, a greater surface area is provided for particle attachment and biological activity. Sand of effective size 0.3 mm - 0.35 mm is common. Larger grain sizes have been used successfully but in that case, the filter may require a longer ripening period.

Coliform bacteria removal has been shown to be effective with sand beds as shallow as 0.5 m. However, scraping reduces the bed depth and it is advisable to provide about 1 m of sand to allow for periodic schmutzdecke removal.

Although rapid fluctuation in hydraulic loading of any filter is undesirable, increasing filter loading rate of slow sand filters does not significantly impair their performance provided loading rates are maintained within 0.04 - 0.4 m/h.

RAPID GRANULAR FILTERS

General

It is important to recognize the fundamental difference in the functions of the slow sand filter and the rapid sand filter. The slow sand filter removes contaminans from raw water largely by bacterial action. This takes place mainly in the zoogleal mat, of schmutzdecke, that forms on the filter surface. It is assisted by exocellular polymers produced within the filter bed. Indeed, a slow sand filter that has not "ripened", i.e. in which biological activity is not established, will not work very well. Once the filter has gone through the ripening period the bacterial action that takes place in the filter will break down organic matter and remove disease causing organisms. Slow sand filters are not well suited to waters carrying a large suspended solids load or to those with high algae counts.

In the case of the rapid sand filter, the role of particulate removal is shared with chemical coagulation, flocculation and sedimentation. Even in the case of direct filtration, it is these same mechanisms of coagulation, flocculation and sedimentation that are largely responsible for particulate removal. The difference in this case is that these mechanisms take place within the filter rather than in separate unit process structures. The rapid sand filter is of virtually no use in removing uncoagulated colloidal clay particles. Such particles will often chelate bacteria and viruses protecting them from the germicidal action of chlorine or other disinfectants. It will not remove disease causing organisms from untreated water. Rapid sand filters operated without chemical pretreatment cannot be relied upon to protect public water supplies. On the other hand, rapid sand filters are excellent for removing colloidal matter that has been destabilized by metal salts such as alum or ferric chloride.

Nature of Colloidal Suspensions

In order to have an appreciation of how rapid filters work, and the role they play with other unit processes in water treatment, it is necessary to understand something about the nature of colloidal dispersions.

When finely divided sand is stirred in a sample of water, the water becomes very cloudy and inhibits the passage of light. If allowed to stand quiescent, however, the mixture becomes clear again as the sand particles settle to the bottom of the sample. The mixture is a suspension of sand in water. Salt or sugar, on the other hand, dissolve completely in water without any interference with the passage of light. Such a mixture is true solution. Colloidal dispersions lie somewhere between a suspension and a true solution; tending to be closer to the latter. Colloidal dispersions abound in nature and are not limited to solids dispersed in liquids. Fog and smoke are examples of liquid and solids dispersed in a gas; milk is a dispersion of two liquids; a ruby is an example of a solid dispersed in a solid. In water treatment we are concerned primarily with colloidal dispersions of clay (complex alumino silicates) in water.

Many of the properties of colloidal dispersions relate to the size of the solids dispersed in the medium. Colloidal particles are usually defined as those having at least one dimension between $1m^u$ and $200\ m^u$ ($1m^u$, or 1 nanometre, is one millionth of a millimetre). Generally speaking, particles less than 1m(u) do not scatter light. Hence, true solutions do not exhibit turbidity. Colloidal dispersions, however, do scatter light, the well known Tyndall effect, and therefore contribute to turbidity. In fact, turbidity of colloidal dispersions is measured by the amount of light scattered at 90^0 to the water surface using a nephelometric turbidimeter.

Clay particles in a colloidal dispersion are in constant random, or Brownian, movement (named after the British botanist Robert Brown) caused by bombardment of the particles by water molecules. This constant movement is an important factor in their remaining indefinitely in suspension. Another, and greater, factor is the surface active negative charge carried by clay colloids. The source of this charge is the preferential adsorption on the colloid surface of a particular ion from the water. The negatively charged clay colloid is surrounded by positive charges in the dispersion (aqueous) phase resulting in an electrical double layer between solid and water. It is this diffused electrical double layer which overcomes the novmal gravitational attraction between solid particles (van der Waals forces) and is primarily responsible for maintaining the solid in suspension.

Destabilization with Metal Salts

Colloidal dispersions of clay in water are extremely stable and will remain in suspension indefinitely. In order to allow settling to take place, the colloids are destabilized with metal salts such as aluminum sulphate and ferric chloride. This process is known as coagulation and it is an essential prerequisite to rapid granular filtration.

When aluminum sulphate is dissolved in water, the metal aluminum ion immediately hydrates, coordinating six water molecules, and forming $Al(H_2O)_6^{3+}$. This aquometal ion then reacts forming several positively charged hydrolysis products followed by solid aluminum hydroxide. Formation of these intermediate complexes is extremely fast; from microseconds to less than one second. Rapid mixing is necessary to ensure complete dispersement of the coagulating chemical in the water and thorough mixing of the resulting intermediate hydrolysis products with the particulate matter held in suspension.

The positively charged hydrolysis products, of which the predominant are $AlOH^{2+}$,$Al(OH)2+$ and $Al_8(OH)_{20}^{4+}$, are readily adsorbed on the surfaces of the colloids held in suspension in the water. The negative charge on the colloids is thus effectively neutralized allowing settling to take place. This adsorption neutralization takes place in less than one second and is the dominant mode of coagulation at low alum dosages (4 mg/L or less).

At alum dosages of 15 mg/L and higher, sweep flocculation is the primary mechanism of coagulation. Gelatinous aluminum hydroxide flocs are formed which enmesh the collloidal particles and settle them out of suspension. The aluminum hydroxide precipitate which is essential for sweep flocculation forms in 1 - 7 seconds after addition of aluminum sulphate to the water.

Critical Rapid Mix Time

The critical rapid mix time will vary depending on the dominant mode of coagulation employed. Where adsorption destabilization is the major factor, rapid mixing must be completed within one second of addition of the coagulating chemical. In the case of sweep flocculation, the mixing time can be several seconds to one minute allowing full hydrolysing of the metal ion and good initial floc formation.

Mixing Intensity

The rate at which particles in a volume of fluid will collide is proportional to the number present, their diameter and the velocity gradient (shear rate). The velocity gradient, the difference in velocity between two streamlines in the fluid being mixed, varies with time at any one point in the fluid and with distance from the mixing agitator. An average value of the velocity gradient, the temporal mean velocity gradient, G, is used in mixer calculations.

Two basic types of rapid mixer are used: the back mix reactor and the plug flow, or in-line mixer. The choice of mixer is largely determined by the type of coagulation being employed.

Back mix reactors employing mechanical agitators in a compartmentalized tank and having retention times of up to one minute are often used where the dominant mechanism for particulate removal is sweep coagulation. Rapid mixing of the intermediate hydrolysis species with the colloidal particles is not essential in this case. They are well suited to conventional water treatment where flocculation and sedimentation prior to filtration follow rapid mixing. High intensity mixing and a one minute retention time allow for full hydrolysing of the aluminum ion and good initial floc formation.

Where adsorption destabilization plays a major role in particulate removal, as is often the case, for example, in direct filtration, mixing of the intermediate hydrolysis species with the colloids must take place in less than one second. High intensity plug flow mixing is required to ensure effective coagulation in such a short time period. A common type of plug flow mixer is the in-line blender comprising mechanical agitators installed in the pipe or conduit and providing a retention time of a few seconds or less. An alternative type of plug flow mixer finding increasing acceptance in Europe is the cascade hydraulic mixer.

Values of G for rapid mixers vary from 3000 - 4000 sec^{-1} for plug flow mixers to 700 sec^{-1} for back mix reactors.

Mechanisms of Filtration

Straining, simple entrapment of solids in the interstices of the filter grains, plays virtually no part in filtration of potable water. If it did, there would be little need to chemically treat the water prior to filtering. Why straining is not a part of filtration can be understood by considering the sizes of the particulates to be removed in relation to the size of filtering media used. As we have previously seen, clay and bacteria exist in water as colloidal dispersions. The clay colloids themselves often encapsulate bacteria and viruses. These colloids are measured in nanometres (millionths of a millimetre). Commonly used filter media has an effective size of between about 0.5 and 1.5 mm. Untreated colloidal dispersions simply pass straight through rapid granular filters.

It is generally agreed that the three mechanisms involved in filtration are transport, attachment and detachment. Transport mechanisms move the particle through the bulk of the fluid within the filter pores so that it comes into close proximity to or in contact with the filter grains. Attachment mechanisms retain the particle on the surface of previously attached particles or the filter medium itself. Detachment mechanisms result from shearing action of the fluid removing part of the attached particulate matter from other particulates or from the surface of the filter grain to be deposited deeper in the filter bed.

Transport mechanisms include gravitational settling, diffusion, hydrodynamic forces and interception. These mechanisms are influenced by the size, shape and density of the particulate matter; the fluid temperature and viscosity; the filter grain size and filtration rate.

Attachment mechanisms are due to physiochemical and molecular forces. Attachment is often improved by molecular bridging of polymers and metal hydroxides. Van der Waals forces, surface tension, electrostatic surface interaction and specific adsorption all play a part. Attachment mechanisms are greatly influenced by chemical pretreatment and the chemical characteristics of the water and the filter media.

Particulate matter removed from the water by transport and attachment mechanisms builds up as sediment in the filter pores and is held in equilibrium with the shearing action of the fluid. As the pores become clogged, the interstitial velocity increses, detaching some of the particulates which are then transported deeper into the filter to be attached to other sites on the filter grains. Eventually all available filter capacity is used up. Further filtering at this stage would lead to turbidity breakthrough. Similarly, disruption of dynamic equilibrium by a sudden increase in filtering rate would cause a deterioration in filtrate quality. When the filter capacity is exhausted, the filter is taken out of service for backwashing.

Backwash Systems

The filtering cycle is normally terminated when the head loss across the filter reaches about 2.5m. Cleaning is accomplished by reversing the flow at a rate high enough to fluidize the filter medium, in the order of 35 m/h, and discharging the wash water to waste. Bed expansion is necessary to increase the pore space within the filter, thereby allowing easier transport of the dislodged particulates. Backwashing is continued for a period sufficient to remove all previously entrapped particulate matter.

Two major factors are involved in dislodging the entrapped or adhering suspended matter from the filter grains:

- the shearing action of the backwash water, and
- the scouring action of filter grains rubbing together.

The former is achieved by the high backwash rate during bed expansion whereas the latter requires essentially no bed expansion and, hence, a low backwash rate. A three stage backwash system in which the filter is first washed at a low rate followed by a high rate wash and then another low rate is often employed. The initial low rate uses the filter grain scouring effect while the high rate makes use of the backwash water shearing action. The final low rate backwash is employed to restratify the filter bed.

Early rapid sand filters were often backwashed at a rate high enough to expand the bed by 50%. Since the advent of dual media filters and coarser filter material backwash rates, and hence bed expansion, have tended to be reduced.

Filter bed fluidization occurs when the upward flow of the backwash water reaches a velocity sufficient to suspend the grains in the fluid. At this point, the friction force exerted by the water on the filter grains is equal to the weight of the grains in water. Further bed expansion and higher backwash rates do not improve the cleansing effectiveness.

The shear force exerted by the washwater on the filter grains during backwashing is directly proportional to the viscosity of the water. Consequently, the washwater rate must be increased during summer when the water temperature is higher. To achieve the same bed expansion with a water temperature of 25°C requires between 30 and 50% higher rate of backwash than when the temperature is 0°C.

The shearing action of the washwater alone will be insufficient in many cases for grain sizes less than 0.8mm. Auxiliary backwash means are employed in most cases to improve the backwashing efficienty. These include surface wash, subsurface wash (at the interface of the sand anthracity layers) and air scour. Surface wash is the most common means employed in North American plants to augment filter backwashing.

Rotary surface wash agitators are used extensively in Canada and elsewhere in North America as well as overseas. Their prime purpose is to break up the accumulation of solids in the upper layer of the filter before the bed is expanded during backwashing. Without surface agitation these solids tend to sink into the filter during backwashing forming "mudballs". Such mudballs are very difficult to remove without taking the filter completely out of service and removing the media. Surface wash agitation is normally started at the beginning of the backwash cycle and maintained during the low rate part of the backwash.

A variation to the surface wash agitator, used with dual media filters, is the subsurface agitator installed just above the sand-anthracity interface. Because of the mixing of the media at the interface filter clogging tends to occur at this point. The subsurface wash agitator ensures thorough cleaning of the interface zone and hence protects against mudball formation here.

European practice favours simultaneous air water wash for 2 to 5 minutes followed by 3 to 5 minutes of water wash at rates sufficient to produce 10 - 30% bed expansion. In this case particle removal is achieved by both abrasion and hydraulic shear. Many authorities consider air scour to be mandatory when polyelectrolytes are used. When air scour is used in Canada or the United States, it is almost invariably used alone and prior to backwashing.

Direct Filtration

Direct filtration is defined in the 1980 AWWA Filtration Committee (Direct Filtration Subcommittee) report as a process train sequence in which filtration is preceded by coagulant addition, some flocculation but no settling. This definition acknowledges that some flocculation always follows coagulation. Perikinetic flocculation caused by Brownian motion of the colloids occurs instantaneously with coagulant addition and is directly related to the water temperature. Orthokinetic flocculation results from bulk motion of the water and turbulence within the water as it is conducted from the mixing chamber to the filters.

A distinction is often made between direct filtration and contact filtration. In the former case, most of the orthokinetic flocculation is provided by separate flocculators. In the latter, separate flocculators are not employed and all flocculation takes place due to hydraulic forces within the conduits, channels and the filter bed.

When separate flocculators are used with direct filtration, they are designed to produce a dense pinpoint floc. The floc penetrates the coarse granular filter media where further flocculation and particulate removal take place. In contact filtration (no separate flocculators) coagulant addition and rapid mix take place immediately ahead of filtration. The microflocs which are formed penetrate deep into the filter bed. Orthokinetic flocculatin takes place within the filter at a much higher rate than normally afforded by conventional flocculators. In both direct and contact filtration, all removal of particulate matter takes place within the filter.

Because there is no sedimentation step in the direct filtration process and all impurities in the raw water are removed entirely by the filters, the filter units are generally designed with large storage capacities. This is accomplished by using relatively coarse media and/or increasing the filter depth. Recent trends appear to favour deeper filters than are normally associated with conventional filtration. Within reasonable limits, coarse filter beds can produce the same quality filtrate as finer beds provided a polymer is used as a filter aid. The larger pore sizes within the filter permit greater penetration of floc, and use of the full bed depth for storage.

Dual media filters with coal effective sizes in the 1.0 and 1.2 mm range and sand of effective size 0.45 - 0.5 mm are commonly employed for direct filtration in Canadian and U.S. plants. In recent years, coal with an effective size of up to 1.7 mm has been used where algae have caused filter clogging problems.

Filters employing 2.4m of 2mm effectie size sand have been recommended for direct filtration as the result of pilot studies undertaken by the Los Angeles Department of Water and Power. The Los Angeles Aqueduct Water Treatment Plans employs 2 m of coarse anthracite following preozonation and microflocculation. These filters are operated at extremely high rates. Single medium filters comprising 900 mm of sand with an effective size of 2.4 mm have been tested successfully in a pilot plant for Brasilia.

Filtration rates are selected on the basis of raw water characteristics, required effluent quality and desired filter run length. In general, filtration rates in the range of 8 to 15 m/h are used. A rate of 10 to 12 m/h is common.

Backwashing procedures for filters used in direct filtration are similar to those used for conventional treatment. Frequency of backwashing can be expected to be greater for direct filtration. European practice favours combined air-water backwash. Water backwash with surface agitators is more common in North America although a number of plants use air scour preceding the backwash. The usual backwashing rate is 37.5-60 m/h.

The presence of algae in the water to be treated is one of the most frequently cited limiting factors for the application of direct filtration. It is difficult to establish a threshold number for algae because filter performance is also dependent on the actual species of the algae present. A maximum number concentration of up to 2000 ASU/ml has been reported.

Experience at the Lake Huron (Grand Bend) filtration plant since it was placed into operation in July, 1987 indicates that diatom concentrations in the order of 1000 ASU can cause 60% decrease in the filter run length for dual media filters (0.9mm effective size coal with a uniformity coefficient of 1.7; 0.5 mm effective size sand with 1.8 uniformity co-efficient). Pilot plan and, later, full scale tests showed that by increasing the coal size to 1.5 - 1.7mm diatom counts as high as 5000 ASU could be handled satisfactorily while maintaining a minimum filter run length of 12 hours.

Problems were subsequently encountered, however, with meeting a turbidity limit of 1.0 TU during periods of high turbidity (approximately 50 - 200 TU) and even at a raw water turbidity of 13.3 TU. High diatom counts in the raw water reduced the filter runs to 4 - 6 hours and resulted in 26% of the treated water being used to backwash the filters. Subsequently, high rate sedimentation basisn equipped with plate settlers and sludge scrapers were constructed ahead of the filters.

Although Toronto's R.L. Clark Filtration Plant was not designed for direct filtration, it operates in this mode due to the high overflow rate in the sedimentation tanks. This is particularly true at flows over 450 ML/d. Prior to 1986 substantially less than 50% of algae were removed in the sedimentation stage during times of high algae concentration in the raw water. Filter runs as short as four hours for single medium filters and ten hours for dual media filters were recorded. In the latter half of June 1988, algae concentrations up to 1400 ASU coincided with high water demands and resulted in filter runs as short as 6-8 hours for several days. Recent changes to the operating procedures have allowed more algae to be removed in the sedimentation basins and, consequently, have led to longer filter runs. It is doubtful that reasonable filter runs could be obtained if the plant were operated in the true direct filtration manner at times of high algae concentration, i.e. operated without sedimentation.

Direct filtration is generally favoured for low turbidity low colour water where adsorption destabilization is the major factor in coagulation i.e. at low coagulant dosages. Lake Ontario water can be treated successfully by direct filtration in most cases. It is sometimes difficult, however, to meet turbidity guidelines of less than 1.0 TU when the pH of the raw water is above about 8.2. Alum coagulation at this pH is poor an pH suppression with sulphuric acid would be needed. Water supplied to many southern Alberta communities through the St. Mary River Irrigation system would probably be a good candidate for direct filtration.

Benefits of direct filtration include lower chemical dosages and lower sludge volume generated as well as lower initial capital cost. Consideration must be given, however, to providing some means of reducing the algae loading to the filters during algae blooms in the raw water. Dissolved air flotation is an effective means of achieving this reduction. As with all water treatment methods, pilot plant testing should be undertaken before embarking on the design of a direct filtration plant. Unfortunately, we engineers have not yet been able to convince regulators of the prudence of this step.

Filter Aids

Synthetic polymers were introduced to the water treatment field in the early 1960's and quickly replaced activated silica which had long been the leading coagulant aid. Their use as filter aids soon followed as interest in high rate filtration using dual and mixed media filters increased. The need to meet more stringent turbidity limits and the acceptance of direct filtration as a viable water treatment process has caused many plant operators to look to the use of polymers as an essential part of modern day water treatment.

In spite of the success of polymers used as filter aids, their selection for water treatment applications is still often based on the results of coagulation studies performed with a standard jar test. Although jar test data do appear to be suitable for an initial selection of the optimum dose for coagulation, they are not always reliable in predicting filtration performance. In fact, the use of jar test results can often be misleading when trying to apply the same results to filtration.

Polymer, pH, and ionic strength of natural waters are generally the three most important variables that can be used to improve the efficiency of filtration by modifying the surface characteristics of either the suspension, filter, or both. In practice, however, pH and ionic strength are not the variables routinely controlled during the course of water treatment. The improvements in filtration that are obtained from a proper selection of polymer dosages are:

1. increased rate of deposition
2. increase bed penetration and better utilization of the entire bed depth, and
3. reduction in high initial head loss resulting from filter surface clogging
4. control of filter breakthrough.

The effect of rapid mixing is another consideration in evaluating or selecting a polymer for use when direct filtration is employed. Although insufficient turbulence during destabilization can be corrected by increasing flocculant dose, this often leads to shorter filter runs and increased backwashing.

TROUBLE SHOOTING
GROUNDWATER TREATMENT PLANTS

- Iron/Manganese/H_2S/Organic Colour -

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Introduction

Ground water containing iron and frequently manganese is common in Western Canada. Target levels for reduction are becoming more stringent, typically 0.3 mg/L maximum for iron and 0.05 mg/L maximum for manganese. These levels, particularly for manganese, are frequently very difficult to attain, in many plants. Many factors are involved as this paper describes.

Hydrogen sulfide removal is discussed, and problems with organic colour.

Iron:

Typically present in well water as ferrous bicarbonate and may also be present as ferrous silicate. On contact with air, oxidizes to insoluble ferric hydroxide which imparts characteristic red-brown stains to fixtures, laundry, etc. and encourages iron bacteria growth. Level of removal required < .3mg/L and preferably lower. Not a health hazard cosmetic.

Manganese:

Typically present as manganous bicarb. May also be tied in with organic acids and as manganous silicate. On oxidation causes black stain problems. Level or removal required to .05 mg/L maximum. Not a health hazard - severe nuisance/cosmetic problem.

H_2S:

Typical rotten egg odour, noticeable in extremely small amounts. Removal required totally.

Colour:

Imparted by humic/fulvic/tannic acids from passage of the water over decaying vegetation. Treated water should have < 10 colour units (15 colour units will engender some complaints, over 15 will cause many complaints). Cosmetic problem rather than health hazard. High colour also makes it difficult to maintain residual sterility as colour components typically have a Cl_2 demand.

BASIC MODES OF REMOVAL

Iron:

Oxidation with air (or other oxidants such as Chlorine/ClO_2/H_2O_2, Ozone, Permanganate) with removal of insoluble oxidized iron by filtration or coagulation/filtration combinations.

Oxidation with air: can be very slow, increasing with increasing pH. Oxidation rates much inhibited with iron organic complexes.

Oxidants other than air: may be tricky. Pre chlorination in considerable disfavour if organic content is significant.

ClO_2: too complicated for smaller plants.

H_2O_2: frequently a winner.

Ozone: expensive but very effective (watch out on processes using MnO_2 contact oxidation media, eg. Greensand. Takes Mn up to permanganate).

Permanganate: strong oxidant. Danger of pink colour on slight overfeed. Also $KMnO_4$ contains approximately 35% manganese which on overfeed can easily impart manganese above .05mg/L solving one problem and replacing with another.

Combination processes of aeration/filtration in one filter eg. "Layne" filters are fairly common.

Another popular mode is contact oxidation type processes with or without pre oxidation or oxidant addition Cl_2/$KMnO_4$/air where the contact oxidation vessel serves as a filter also. Eg. Greensand/BIRM/Pyrolox process.

Manganese:

Typically associated with iron though sometimes found alone. Manganese is removed by the same unit process described above for iron, though there are several points of caution:

Manganese is much slower to oxidize than is iron. Rate of oxidation is also a function of pH, increasing with increasing pH levels but requires higher pH than iron.

The oxidation product of manganese is manganite ($MnOOH$) which has a tendency to form a negatively charged colloid which creates quite a filtration problem to remove (iron forms the positively charged $Fe(OH)_3$ which is much easier to coagulate and filter out than is manganite).

More drastic measures may be required for manganese removal than for iron in the area of stronger oxidants, more reaction time, pH adjustment, use of filter aids etc.

Contact oxidation processes using MnO_2 are frequently the best fit for manganese removal. The MnO_2 seems to loosely adsorb manganite.

Hydrogen Sulfide

Difficult to remove by aeration as pH is seldom favorable (requires low pH).

Air oxidation is slow and often leads to the formation of polysulfides which themselves have a taste and odour - different from H2S but still unpleasant.

Best bet is usually chlorine. Peroxide fine also.

Permanganate is often a problem with H_2S as its reduction product is manganite so it may be no bargain to exchange H_2S smell for a brown discoloration due to colloidal manganite.

Contact oxidation processes work well with small quantities of H_2S, but larger quantities of H_2S can lead to a "sifting through" of manganese sulfide.

Colour

Is more often associated with surface water treatment but is common enough with well water, often requiring, when iron and/or manganese is present, a two stage approach.

Colour removal is best effected in a low pH regime, the opposite of iron/manganese oxidation.

Techniques of removal are many, including alum or PAC coagulation followed by manganese removal (iron will usually be removed in the coagulation process).

On smaller plants iron/manganese removal might be followed with granular activated carbon columns for final colour removal.

Some plants successfully remove colour by in-line clarification or direct filtration with regular filters or upflow contact clarifiers and polish Manganese removal afterwards.

Troubleshooting

Troubleshooting quite obviously means looking at a plant designed to do something or a number of things which is not now doing very well maybe even originally was not so super. As can be seen from the many variations of contaminant combinations and process and control complexities, and equipment designs, this can be a pretty tall order.

Initial troubleshooting procedure is obvious and straight forward eg. is the basic process and design O.K.? Hydraulics O.K.? Has the raw water changed in any significant way? Is operation satisfactory? Look at frequency and rate backwash and P at run termination and what is clean bed headloss after backwash.

Surprisingly many plants are flying blind in the sense that no flow measuring devices have been provided to measure backwash rate of flow, or even service rate of flow, and no means of throttling backwash rate-of-flow.

We will now discuss, specifically, problems with greensand filters and the like, and discuss state-of-the-art technological improvements in contact oxidation processes.

MOST COMMON PROBLEMS

GREENSAND FILTERS

Greensand has been used for decades. It is made by using the mineral glauconite as a particulate base. Glauconite has a greenish colour and has some ion exchange properties. Greensand is prepared by treating sized glauconite with manganese salts so the ion exchange capacity is used up with manganous ion. Then the manganous ion is oxidized with permanganate to form a thin coating of MnO_2 which is strongly oxidizing and will oxidize ferrous and manganous ion to the insoluble higher oxides which are then removed by a combination of adsorbency and filtration. When the oxidative capacity is used up, the material can be regenerated back to the MnO_2 form with permanganate.

Greensand has very little MnO on it, so has little capacity for iron and manganese. Operating in the direct mode requires frequent regeneration so use in this manner is usually limited to small domestic applications.

On larger Greensand plants, preoxidation of iron and manganese is practiced, frequently using permanganate as preoxidant, frequently chlorine and permanganate, sometimes chlorine alone, sometimes peroxide, sometimes aeration and permanganate. Ozone is tricky here as ozone is such a powerful oxidant it will take manganese right up to the +7 valence permanganate causing pink water problems.

The theory thus carries on that by positively preoxidizing iron/manganese the greensand filter acts a s a filter, in removing the insoluble precipitates and has buffering capacity to oxidize residual ferrous and manganous ions if preoxidant control is not so great and will be continuously regenerated by and remove excess permanganate thus protecting against pink water break through.

To enhance filter performance, usually a layer of filter coal overlies the greensand to provide a dual media function.

This all sounds like a straight forward process but often turns out to be very troublesome indeed.

A very common problem is to find that the filter has poor backwash distribution. Many of the filters used have hub lateral backwash distributors which by their very configuration give rise to uneven upward flow of backwash water. This creates "mudballing" and short circuiting of incoming water flow with steadily declining performance. Can be corrected to a degree with careful gravel diffusion layering but often not to the degree required.

Strainer type false bottom filters can be troublesome as the strainers create a spouted bed effect, again resulting in short circuiting, through the channels created.

Poor underdrain design is likely the No. 1 problem in most of these plants.

Again on underdrains, often greensand is used with no gravel layering over strainers of various types. Greensand is quite fine and a good percentage can pass through the strainers and on backwash the strainers may plug from the underside. Strainer plugging is common in these filters and overall performance becomes terrible. Graveling will correct to a degree, but sometimes a better underdrain retrofit is required.

Another common problem is the use of a filter coal layer of a size that is incompatible with the fine greensand. Then much of the greensand mixes with the coal and vice versa. The coal is there to function as a flocculation zone/roughing filter. The ferric hydroxide formed by preoxidation and the manganite need the agglomerating effect of a flocculating zone to become filterable. If coal sizing is wrong much of this effect is lost.

Common problem again is failure to recognize that greensand is quite friable. As supplied it contains a lot of fine material so backwashing and skimming is mandatory - typically remove at least an inch for every foot of greensand depth. Often this is not done, and often is very difficult to do as access to small pressure filters is generally poor. If so, the greensand should be dry screened for fines removal before loading. If fines remain the material will exhibit a high clean bed headloss. If taken over 8-9 psi headloss, greensand will start to crush, so failure to remove fitness means short runs to allowable maximum headloss. What usually happens is that recommended maximum headloss is then exceeded to lengthen runs and progressive crushing of greensand occurs.

Again the filter coal layer function is also one of roughing filtration and to try to take the brunt of headloss across the coal rather than across the greensand. Poor coal size selection obviates the function.

Another problem with disrupted greensand filters follows from the obvious fact that potassium permanganate contains approximately 35% manganese. Takes very little permanganate to sift through the filter to exceed the .05 mg/L target maximum of manganese.

Low pH operation [, 6.5] will strip the greensand of manganese.

Greensand will oxidize low levels of H_2S but higher levels than 5 mg/L approximately may result in sifting through manganese sulphide.

Greensand will do nothing for colour removal, though permanganate addition might.

In passing we should mention BIRM media filters. BIRM, which stands for "Burgess Iron Removal Method" uses a particulate base of light porous materials eg. pumice. The base material is treated with manganous salts to near saturation, the manganous ion being then oxidized to MnO_2 with permanganate or caustic/air addition.

The material thus has considerable similarity to greensand though the suggested mode of process operation differs. The recommendation is to introduce air ahead of the BIRM filter, no other chemicals, and the BIRM by catalytic action will speed up the oxidation of iron and manganese with their removal on the BIRM by filtration.

Reliance is thus on a good operation as a filter and all the problems of backwash distribution mentioned with greensand, apply here as well.

Further no filter coal flocculation zone is used nor can one be used as the specific gravities are too close, so the BIRM has to do all the work. It is not the strongest material in the world and crushing is a problem.

It is not recommended if H_2S is present or if organics are present.

Aridsorb [synthetic zeolite particulate base treated with Mn++ etc. like greensand] is another material used in the past as a greensand equivalent with certain advantages claimed, in particular, low gravity requiring low backwash rates. There were many problems with aridsorb, but as it is no longer available there is little point in much discussion.

Occasionally there are raw waters with iron only - no manganese - where a simple dual media or single media filter with preoxidation will work well. Seldom is this true if manganese is present.

We have seen the complexities of greensand filters and it is easy to conclude that this is a tricky process to use with many areas that can contribute to poor performance. Similarly with BIRM

The urgent need for a simple, reliable, positive process for iron and manganese removal has been met with the new "PYROLOX" contact oxidation process. This process will undoubtedly be used to retrofit many existing greensand and other types of plants and be the process of choice for new plants.

It is a simple process and very effective, based on the use of "PYROLOX" a very high grade, impurity-free, form of sized particulate manganese dioxide, a strong oxidizing agent.

This is a new method in a sense that it is only fairly recently that a high grade manganese dioxide in suitable particulate sizes has become available in the U.S. and Canada. It is an old method in that the concept has been in use in Great Britain for decades.

As we have seen, greensand, BIRM, and aridsorb all rely for their function on a very thin layer of manganese dioxide while the Pyrolox process has relatively massive amounts of MnO_2 in contact with the water.

The process has considerable flexibility. On small applications a discrete layer of Pyrolox or a blend with filter sand would be used in the filter, with no preoxidation. The Pyrolox would slowly be used up but has such a high capacity for the normally low level iron and manganese that top-ups would be very infrequent.

If raw water pH is low eg. <6.5 a topping layer of sized calcite is used to elevate the pH a bit and speed up iron/manganese oxidation.

Some H_2S can be removed. If H_2S is present above 5 mg/L it is recommended that this be reduced by preoxidation to avoid some potential "leakage" of manganese sulfide.

On larger plants air addition ahead of the Pyrolox filter is desireable to enhance oxidation and conserve the media. Permanganate addition is not necessary. If much iron is present it may be desirable to provide a flocculating/roughing filtration filter coal zone over the Pyrolox. Ferric iron and manganite are only loosely adherent and are backwashed out of the filter. The high density of Pyrolox allows high backwash rates without media loss for thorough cleaning of the filter. Proper filter design is important as it is with any process, to ensure good backwash distribution.

As with any process whatever it is wise to run simple pilot column tests to establish parameters for the design or retrofit. This is a pretty obvious comment but an astounding number of plants are designed and built with no pilot test work. It is particularly important if the iron and manganese are bound or partially bound as organic complexes are much longer than normal times are required for oxidation of complexed iron and manganese.

THE EFFECT OF TURBULENCE ON TURBINE WATER METERS

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Since the early 1970's AWWA Class 2 turbines have gained an increasing acceptance among utilities. This support has been gained mainly due to the highly accurate nature of the turbine, it has fewer moving parts than a compound and it is relatively easier to perform maintenance. It also helps if the compound is higher priced than the turbine. It also has a limitation. It is the purpose of this report to show the effect turbulence has on accuracy.

The turbine is used in the following applications:

High Flow/Revenue

Turbines are most typically used where high flow is being measures as in the case of a municipality selling water to another utility or where the flow is of a consistent high nature as in the case of bulk distribution or industrial user. It is not uncommon to hear of a turbine measuring $50,000:00 per month. One bulk station in Western Canada generated nearly $60,000:00 in its first year of operation.

Treatment

More and more treatment plants use turbines as a basis to perform chemical treatment on potable water and the actual control of their system. A typical installation involves a turbine measuring and producing a 4-20ma signal to pace a chemical injection pump.

Replacement of Compounds

Compounds have traditionally been the accepted meter in sizing 2" through 6". Many utilities have now come to examine each installation and make a decision based on flow and maintenance requirements instead of line size. In may instances turbines are replacing the compound. In these applications the utility has opted for long term accurate measurement.

In each of these applications the need for dependable, repeatable and long term accuracy is critical. On a large meter a few percent inaccuracy can result in a revenue loss or an irate customer service problem. In the case of treatment, inaccuracy can impact on treatment costs and make a proper chemical balance difficult.

The turbine is a velocity meter. The speed of the rotor increases in relation to the flow going through the pipe. This in turn is calibrated to the appropriate measurement. Accurate measurement is dependent on a steady consistent flow pattern entering the measuring chamber of the turbine. Turbulence effects the flow pattern entering the turbine. The degree of the effect was the purpose of a study done by Rockwell International Engineering in the mid 1980's.

The study was performed using standard production 6" turbines placed in situations which have been observed in the field. Sixty tests were performed. Each test consisted of doing a series of accuracy tests at various flows and accuracy curves being generated.

The study was set up in 3 steps:

1. Identification of turbulence
2. Measurement of effect
3. Recommendations

Identification

In piping situations the velocity profile of the flow changes as the flow increases. As a rule of thumb at flows less than or equal to the pipe diameter the flow has a "Laminar" appearance. At flows greater than the pipe diameter the flow condition is called "Turbulent". The change is due mainly to pipe wall friction. The change over point from Laminar to Turbulent is the point at which the greatest inaccuracy is found in a turbine. On the accuracy curve this is typically represented by a slight rise in accuracy curve.

It should also be noted that measurement under Laminar conditions is relatively simple. However under Turbulent conditions measurement is much harder. It was under Turbulent flow conditions the study was performed. For the purpose of the study turbulence was identified as low, medium, and high turbulence.

Low Turbulence

Low turbulence can be found in areas where there are elbows, Y's located within 5 pipe diameters of the meter. The effect of these installations is a "Jetting" of water. The resulting flow pattern is an uneven velocity where one side or area of the turbine is receiving water at an increased velocity.

The expected effect of low turbulence is +/- 1.5% deviation from the normal accuracy curve. Additional results are a decrease service life and an increased maintenance factor.

Medium Turbulence

Medium turbulence conditions occur where there are Y's and elbows. This is further complicated by the meter being in a different plane than the pipe run. Eccentric reducers will also cause this effect. The result of such an installation involves "Jetting" similar to what is encountered in a low turbulence condition. This is further complicated by "Swirl" within the pipe. Under a swirl condition the water is turning within the pipe. The expected result of a medium disturbance condition is a deviation of up to +/- 2.5% from the normal curve. Again service life and maintenance are also affected.

High Turbulence

Finally in a worst case, High Turbulence is the result of pumps, butterfly valves, pressure reducing valves and throttling valves located in close proximity to the turbine. It should also be noted similar effects have been found where these devices are located downstream in close proximity of the meter. As in the medium condition, Jetting and Swirl conditions are present. In addition to these there is an effect on the radial velocity. As water enters the turbine under normal conditions the velocity increases due to the effect of the turbine forcing the flow to pass through the rotor in a smaller area, i.e., the outer area of the turbine. This condition is referred to as the radial velocity. Under ideal conditions the velocity increase is equal on all sides of the rotor. Less than ideal conditions could occur where a portion of the strainer is blocked or a rock is lodged in the nose cone of the turbine. The effect is an even greater increase in radial velocity on one area of the rotor. The expected result of a High Turbulence condition is accuracy +/- 3 to 5% and up. Under this condition the service life and maintenance are the most effected.

Measurement

The following are examples of some of the results observed from the tests. Again it should be noted the piping configurations have duplicated what has been found in the field.

Summary Observations

- Strainers change registration
- Effects are more pronounced at higher flow rates
- Strainers can correct an inaccuracy in Low Disturbance installations
- Strainers correct most of an inaccuracy in a medium Disturbance installation
- Strainers diminish some inaccuracy in High Turbulent conditions
- Valving elements upstream are very catastrophic
- Valving elements downstream also have an effect
- Straightening vanes can effect accuracy
- No real predictability

Optimum Large Meter Installation

The Do's

- Strainer
- Straight pipe up and down
- Full opening valve

The Don'ts

- Restricting valves upstream or close downstream
- Pumps close upstream
- T's, Y's, L's, Misaligned flanges
- Eccentric reducers upstream

Corrective Measures

If you have conditions which where turbulence is a factor here are some recommendations:

- Strainers
- More straight Pipe
- Remove or move disturbance generator
- Provide more flow conditioning
  - Tube type Bundles
  - Perforated plate types
  - Combination types

One word of caution, strainers and other flow conditioners will generate additional head loss and must be kept clean as part of a maintenance program.

Finally, the turbine is a highly accurate measurement tool over an extended range. Under proper conditions it will provide for years of dependable service. To achieve the best performance, manufacturers provide installation guidelines. Review these and if they are not included in your system specifications I urge you to amend your specs.

ANAEROBIC AND AEROBIC TREATMENT OF POTATO PROCESSING WASTEWATER

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Canada

Introduction

The past few years has seen more and more pressure being applied to industrial wastewater producers to clean up their wastewater discharges. The various government levels and the public in general have been voicing their concerns that the environment is not a general dumping ground and in many cases, it cannot accept any further abuse. ADI has been involved in industrial and municipal wastewater treatment for many years and in the past 10 years has been viewing anaerobic pretreatment as an economical alternative to many types of treatment systems that are on the market. When high strength industrial wastes are to be treated, the anaerobic-aerobic system is generally a lower capital cost and a lower maintenance cost system compared to many of the all-aerobic treatment systems that are available.

History of Treatment Plant

The original system was constructed in the late 60s and consisted of screening and primary clarification followed by an extended aeration system and a polishing aerated lagoon; Figure 1 presents a schematic of this system. The wastewater being treated was generated by a potato processing plant; the plant also processes peas during the summer for approximately four to six weeks.

The primary sludge removed from the clarifier consisted mainly of potato solids along with a small amount of soil particles. It was either thickened by a vacuum filter into a cake, which was used as a cattle feed, or pumped to sludge ponds located on site; these ponds produced an odour nuisance (which is still the case at the time of writing).

The effluent from the primary clarifier flowed by gravity to an aeration basin that contained an integral clarifier. Aeration was initially supplied by two, 75 hp compressors feeding air to two types of aeration devices, i.e. Helixors^{TM} and perforated tubing. The compressors also supplied air to a series of air lift pumps, in the integral clarifier, that returned the sludge to the aeration basin. There were no provisions to waste sludge from this system. The effluent from the integral clarifier flowed by gravity to a large polishing pond (lagoon #1E) prior to being released to the Barbara Weit River, which flows into Malpeque Bay, located on the northeast side of Prince Edward Island.

The two original 75 hp compressors were rated to deliver 3000 scfm at 7psig. A third compressor, rated at 125 hp and capable of delivering 3000 scfm at 7 psig, was later added to increase the aeration capacity. Also to reduce the load on the Barbara Weit River the outfall was moved downstream, into the estuary of Malpeque Bay.

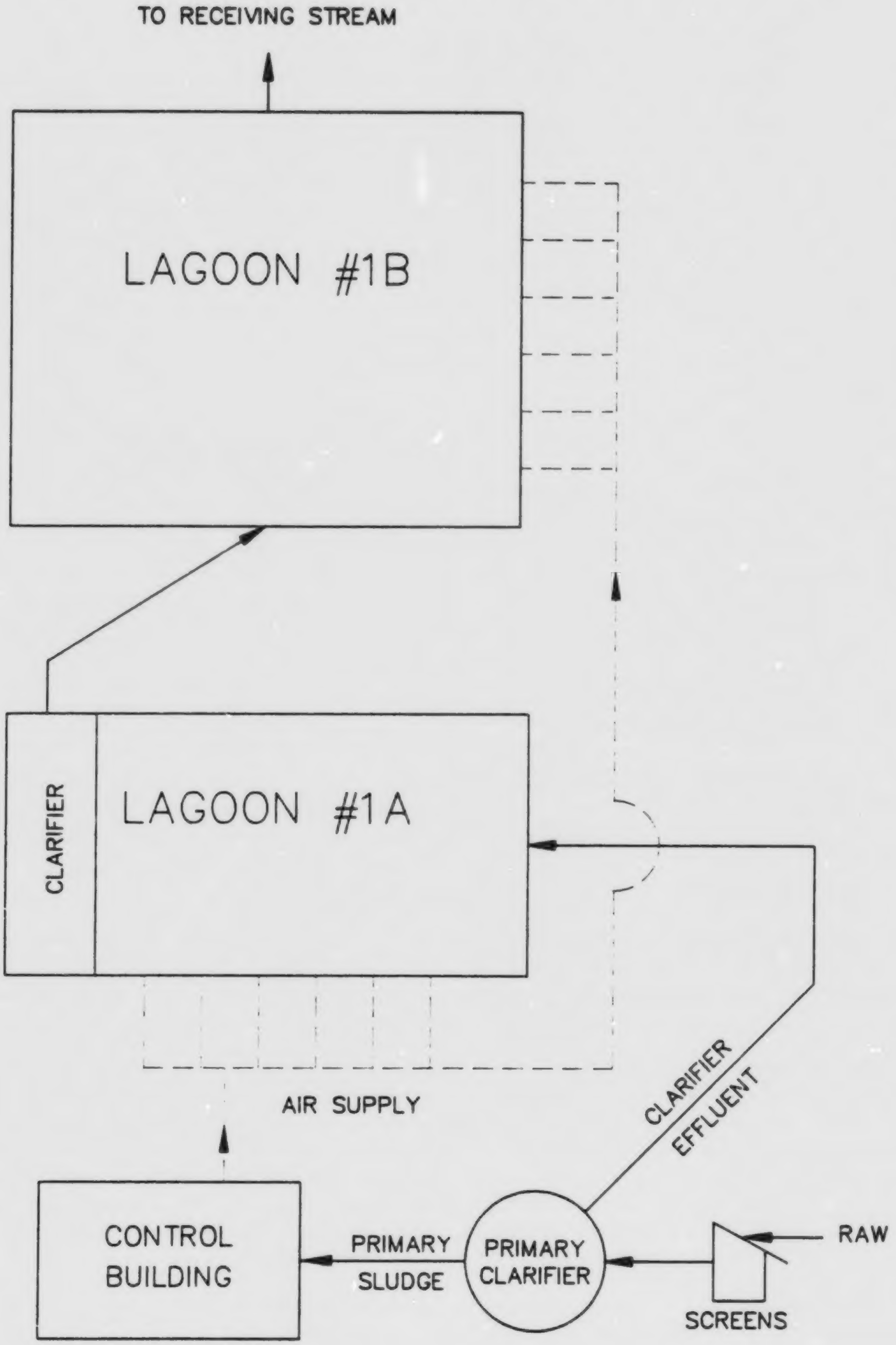

TO RECEIVING STREAM
LAGOON #1B
CLARIFIER
LAGOON #1A
AIR SUPPLY
CLARIFIER EFFLUENT
CONTROL BUILDING
PRIMARY SLUDGE
PRIMARY CLARIFIER
RAW
SCREENS

FIG. 1: SCHEMATIC OF ORIGINAL WASTEWATER TREATMENT SYSTEM

An anticipated increase in potato production and wastewater flow led the Prince Edward Island government (owners of the original system) to approach ADI and request a proposal on upgrading options for the waste treatment plant. The resulting proposal included the addition of a low-rate anaerobic system preceding the extended aeration system. Its purpose was: to pretreat the wastewater with the anaerobic effluent being polished by the ASP prior to being discharged into an estuary; to greatly reduce the amount of sludge that had to be wasted; and, to be a source of fuel (biogas) that could be utilized in the production of electricity that would aid in the financial operation of the waste treatment plant. To confirm design criteria for the proposed anaerobic pretreatment facility, a pilot plant study on site was conducted prior to construction.

The raw wastewater and peel waste are characterized in Table 1; these values were established by the pilot study.

TABLE 1 - RAW WASTEWATER, PEEL WASTE CHARACTERIZATION

Parameter	**Raw Wastewater**	**Peel Waste**
COD	3524 (1287)[b]	140 350 (27 558)
BOD	1464 (613)	62 158 (9 156)
SS	1203 (590)	72 146 (16 793)
VSS	1184 (592)	135 136 (14 932)
TS	2546 (350)	121 621 (14 318)
TVS	1864 (300)	
pH	3.4 to 7.0	
Alkalinity	90 (86)	
VA	975	
TKN	128 (18)	
NH_3-N	33 (6)	
Total P	34 (6)	
FOG	300	

[a] pH in standard units, all others in mg/L.

[b] Values in parentheses represent sample standard deviation

Pilot Plant Study

The pilot plant was operated for approximately six months at loadings and retention times which would simulate or reflect those anticipated in full-scale operation; Figure 2 illustrates schematically the pilot plant apparatus.[1]

The objectives of the study were as follows. evaluate the anaerobic treatability of the raw wastewater with peel waste addition; evaluate the performance of the anaerobic reactor for the purpose of verifying design criteria; evaluate the performance of the anaerobic reactor under stress loading conditions; and characterize the raw wastewater and peel waste for the purpose of predicting combined mixture characteristics.

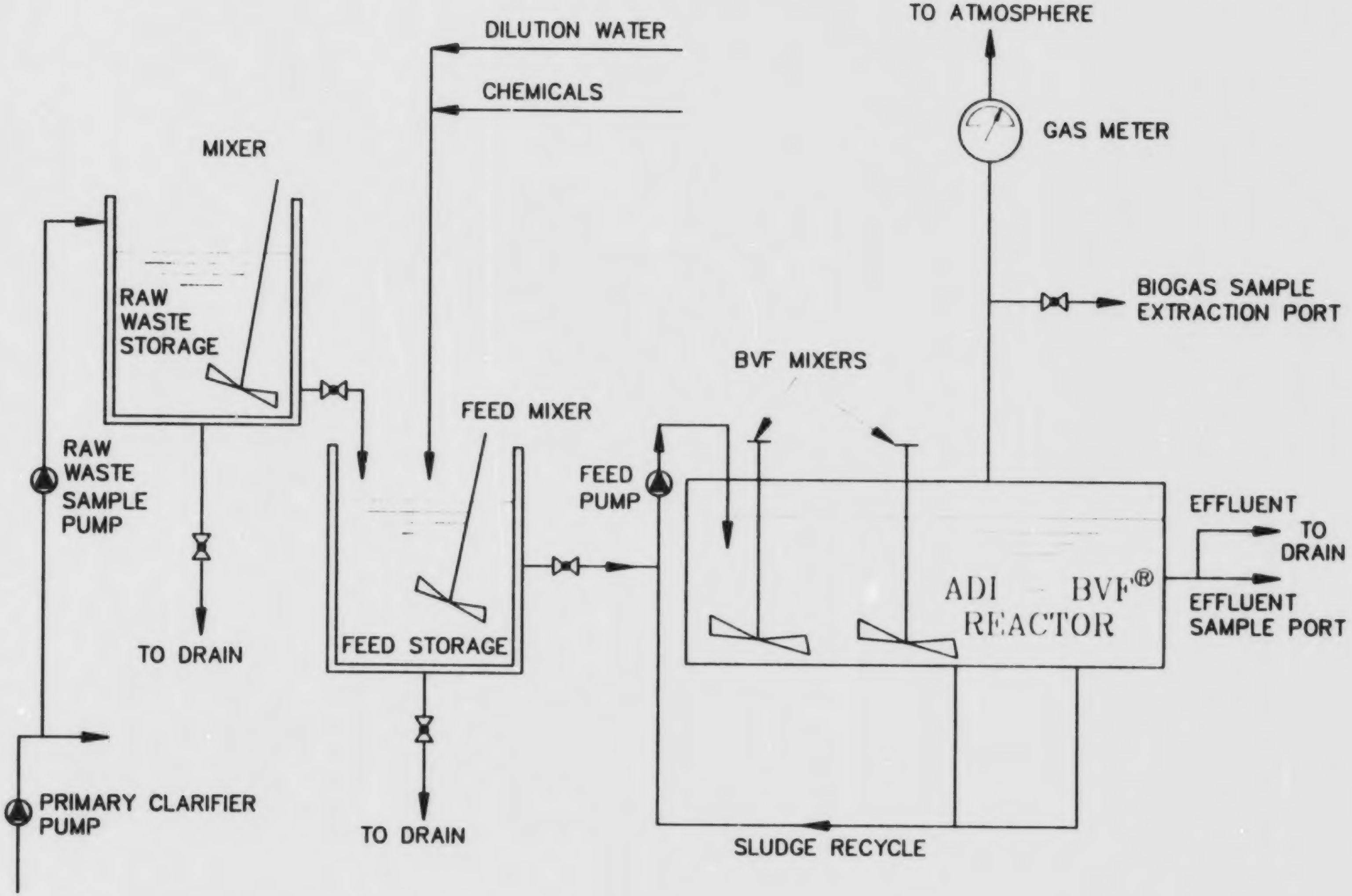

DILUTION WATER
CHEMICALS
TO ATMOSPHERE
GAS METER
MIXER
RAW WASTE STORAGE
BIOGAS SAMPLE EXTRACTION PORT
BVF MIXERS
FEED MIXER
RAW WASTE SAMPLE PUMP
FEED PUMP
EFFLUENT
TO DRAIN
ADI - BVF®
REACTOR
EFFLUENT SAMPLE PORT
TO DRAIN
FEED STORAGE
PRIMARY CLARIFIER PUMP
TO DRAIN
SLUDGE RECYCLE

FIGURE 2: ADI-BVF® PILOT SCHEMATIC

Table 2 presents the results obtained during the pilot study.

TABLE 2 - AVERAGE PILOT PLANT RESULTS

Mode of Operation	Parameter[a]	Influent	Effluent	Percent Removal
Steady-state	COD	7605 (1128)[b]	1304 (196)	82.4
(day 64 to 154)	BOD	3482 (727)	636 (237)	81.1
Temp = 35^{0}C (95^{0}F)	SS	1951 (430)	216 (37)	88.6
HRT = 7.9 d	VSS	1866 (427)	185 (29)	89.7
	pH	3.6 - 7.1	6.0 to 7.0	
	VA	1554 (609)	440 (197)	
	alkalinity	1293 (967)	2277 (551)	
Steady- state	COD	8480 (584)	1281 (213)	84.8
(day 155 to 189)	BOD	4528 (439)	569 (95)	87.4
Temp = 28^{0}C (82^{0}F)	SS	3007 (756)	208 (38)	92.4
HRT = 7.9 d	VSS	2922 (753)	200 (33)	92.5
	pH	5.8 to 7.4	6.4 to 7.2	
	VA	1441 (671)	706 (215)	
	alkalinity	1621 (700)	3150 (424)	
Stress Loading #1	COD	8500	2100	75.3
(day 190 to 194)	BOD	3393	757	77.7
Temp = 28^{0}C (82^{0}F)	SS	3060	239	87.3
HRT = 6.8 d	VSS	3010	342	88.6
Stress Loading #2	COD	8600	1500	82.6
(day 200 to 202)	BOD	4280	733	82.9
Temp = 28^{0}C (82^{0}F)	SS	3690	130	93.7
HRT = 4.5 d	VSS	3610	226	93.7

[a] Temperature expressed in ^{0}C, pH in standard units, all others in mg/L

[b] Values in Parentheses represent sample standard deviation

The following conclusions summarize the findings of the pilot plant study:

1. The potato processing plant raw wastewater (including peel waste supplements and pea wastewater) proved well-suited for anaerobic treatment; average COD, BOD and SS removal efficiencies at 28±2^{0}C operation were 84.8, 87.4 and 92.4 percent respectively;

2. Average COD and BOD loadings achieved during steady-state operation were 1.0 and 0.48 kg/m^{3}.d, respectively; both values exceeded anticipated full-scale design loadings;

3. Average specific methane generation rate, corrected to STP, was determined as 0.35$m^3 CH_4$ (STP)/kg COD removed; the average biogas CH_4 concentration was 61.0 percent; average biogas H_2S concentration was 977 ppm by volume;

4. Removal efficiencies under short-term COD and BOD stress loading of 1.9 and 0.94 kg/m^3.d, respectively, were COD = 82.6 percent and BOD = 82.9 percent; both loadings exceed the anticipated full scale, maximum processing day loadings;

5. Sufficient nitrogen and phosphorus were available in the raw wastewater, such that no nutrient supplement requirements were necessary once reactor maturity was achieved; an average COD/N/P/ ratio of 211/5.3/1 was determined;

6. Alkalinity supplements were necessary to correct influent pH (due to prolonged raw waste storage) and to supply sufficient buffering capacity during start-up operation; a full-scale start-up would require alkalinity supplements for buffering capacity but none would be required for ongoing operation once the reactor reaches maturity;

7. The overall anaerobic destruction of influent solids was determined from a solids balance to be 81.3 percent for total volatile suspended solids and 78.2 percent for total suspended solids.

The following recommendations were made based on the findings of the study:

1. For full-scale design purposes, the following design criteria can be used:

   (a) COD, BOD and SS removal = 85 percent;
   (b) Specific methane generation = 0.32 m^3 CH_4(STP)/kg COD removed;
   (c) Biogas contains 60 percent methane and 1000 ppm hydrogen sulfide;
   (d) Average COD and BOD loadings = 1.0 and 0.5 kg/m^3.d respectively;
   (e) HRT = 8 days;
   (f) Minimum average operating temperature = 28 ± 2°C; and
   (g) Waste anaerobic sludge TS = 4 to 7 percent

2. During start-up, alkalinity supplements should be made to ensure that adequate buffering capacity is available;

3. No nutrient supplements, including nitrogen and phosphorus, are required during start-up or steady-state operation; and

4. Hydrogen sulfide should be removed from the biogas to a concentration which is consistently below 1000 ppm to meet the stipulations set down by the manufacturer of the engine-generators.

The Current Treatment System

Having successfully proven the treatability of the wastewater, which consisted of potato processing wastewater, pea processing wastewater (in season), peel waste, domestic lagoon effluent and waste activated sludge, construction of the BVF™ reactor commenced in the latter half of 1985; start-up of the anaerobic reactor got underway in early 1986.

The reactor is 45 000 m^3 in volume and has 7 840 m^2 surface area—just slightly larger than a football field. It provides and HRT of 8 days and is designed to treat 5 600 m^3/d. The reactor consists of an earthen and concrete basin lined with a flexible membrane and covered with a floating, insulated membrane cover. Within the reactor are two, low-speed mixers and headers used to promote contact between the wastewater and the reactor's culture of anaerobic bacteria. Biogas is collected beneath the floating membrane cover with the rate of draw-off controlled by a programmable controller (PC). The PC also controls other items such as mixers, pumps and biogas utilization equipment.

The mixers within the reactor are capable of providing various mixing modes ranging from no mixing at all to continuous mixing. The mixers can be rotated through 360 degrees in 22.5 degree increments Mixer operation, controlled by the PC, can vary based on either cycle time or real time. The mixers can be removed for maintenance while the reactor remains fully operational.

The effluent from the reactor is polished in the Activated Sludge Plant (ASP). The aeration basin mixed liquor is pumped to a secondary clarifier (the original primary clarifier) where the mixed liquor suspended solids (MLSS) are removed and returned to the aeration basin; a portion of it is wasted to the BVF reactor. The final effluent is discharged to the estuary. The only sludge wasted from the system is waste anaerobic sludge (WANS)--a sludge that can be utilized as a soil conditioner and as fertilizer. Figure 3 schematically illustrates the current anaerobic/aerobic system. Additional details of the plant are furnished in Table 3.

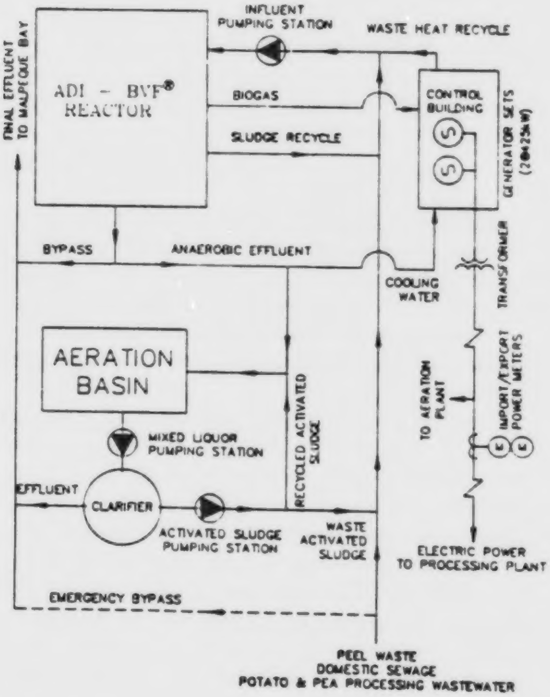

FINAL EFFLUENT TO MALPEQUE BAY
ADI - BVF® REACTOR
INFLUENT PUMPING STATION
WASTE HEAT RECYCLE
BIOGAS
SLUDGE RECYCLE
CONTROL BUILDING
GENERATOR SETS (2@425kW)
BYPASS
ANAEROBIC EFFLUENT
COOLING WATER
TRANSFORMER
AERATION BASIN
RECYCLED ACTIVATED SLUDGE
TO AERATION PLANT
IMPORT/EXPORT POWER METERS
MIXED LIQUOR PUMPING STATION
EFFLUENT
CLARIFIER
ACTIVATED SLUDGE PUMPING STATION
WASTE ACTIVATED SLUDGE
ELECTRIC POWER TO PROCESSING PLANT
EMERGENCY BYPASS
PEEL WASTE
DOMESTIC SEWAGE
POTATO & PEA PROCESSING WASTEWATER

FIGURE 3: CURRENT TREATMENT SYSTEM SCHEMATIC

TABLE 9- FACT SHEET - NORTHEAST ENERGY SERVICES LIMITED
WASTE TREATMENT AND CO-GENERATION FACILITY
NEW ANNAN, P.E.I.

Raw Wastewater	pH: 4-12	
	Temp: 17^0 - 36^0C	
	FOG:<300mg/L	
Average Flow	3800 m^3/d	
Design Flow	5600 m^{3}d average	
Anaerobic Reactor	Volume:	4500 m^3
	Operating HRT:	12d (average)
	Length:	112 m
	Width:	70 m
	Depth:	6.7 m
	Construction:	Earthen basin
		Precast concrete side walls
		HDPE lined
	Operating Temp:	20^0-29^0C; typically 26$\pm$2^{0}C
Aeration Basin	Volume:	7400 m^3
	Operating HRT:	2d (average)
	Length:	73 m
	Width:	46 m
	Depth:	3 m
	Construction:	Earthen basin
		Clay lined
	Operating Temp:	10-26^0; typically 23$\pm$2^{0}C

The biogas produced is a mixture of the gaseous by-products of anaerobic fermentation and generally consists of the following: methane @66%, carbon dioxide at 33%, hydrogen sulfide @0.1 to 0.2% (1,000-2,000 ppm) and others at 0.8 to 0.9%. The biogas is collected by a floating membrane cover that is also designed to control temperature (heat loss/heat gain) and odour. The cover is constructed of high density polyethylene (HDPE) overlaying 12 mm of Ethafoam insulation. The closed-cell construction of the insulation greatly reduces its water absorption rate.

The cover is constructed in such a manner that biogas collected beneath the cover is conveyed through channels to the central biogas collection header.

The rate of biogas withdrawal from the digester varies with the prevailing rate of biogas production. A pressure sensor/transmitter at the reactor transmits a continuous signal to the PC, the signal strength indicating to the PC whether the biogas pressure is increasing or decreasing. The P.C. responds by increasing or decreasing the biogas blower speed so as to increase or decrease the rate of draw-off. This operation attempts to maintain a slight vacuum (1/4 inch water column) under the floating cover; the rate of biogas production or withdrawal are closely matched, and small leaks result in air infiltration rather than biogas exfiltration to the atmosphere.

The system has two, positive displacement, lobe-type biogas blowers, one for duty, the other for stand-by, for biogas flow control. The biogas is discharged to a header on which two biogas conditioning filters are connected. The filters are in use only if the biogas is being utilized by either or both of two engine-generator sets (gen-sets); excess biogas that is being flared is not conditioned. According to the manufacturer, the filters "mask" the hydrogen sulfide to reduce its impact on the operation of the gen-sets.

The gen-sets are Waukesha, 600 hp, V-12, spark-ignition engines driving Kato induction generators. The generators are rated at 425 kW at 1200 rpm. The power first used to meet the needs of the waste treatment plant; any excess is sold to the processing plant that generates the waste. The gen-sets also aid in maintaining a higher temperature within the anaerobic reactor. Effluent from the reactor flows through a shell-and tube and tube heat exchanger on each gen-set. This flow is then mixed with the raw wastewater. The cooling systems for the gen-sets are also tied in to the heating system for the control building.

There are two full-time employees at the waste treatment plant; the plant is only manned 8 hours per day, 5 days per week. The automatic control provided by the PC maintains the system during the unmanned periods. ADI is under contract to provide engineering and administration as well as day-to-day supervision of the plant operation.

Plant Performance

The performance of the waste treatment plant has been steadily improving as the operators became more familiar with the operation, as communications improved between the food processor and the treatment plant, and as the reactor matured. A reactor of this size requires a large quantity of biomass to ensure optimum organics removals and this one was no exception. Seed sludge was not readily available, the main source being a secondary municipal digester in Charlottetown, P.E.I., and it could not produce the amount and quality required to get the reactor in service on schedule. Consequently, it was approximately mid-1988 before the reactor could be considered mature, containing the proper amount of biomass. Prior to this, pH control chemicals were required on a semi-continuous basis to keep the reactor pH within an acceptable range. Since maturity has been attained, pH control chemicals have not been required.

Concomitant with the reactor reaching maturity, biogas recovery has been steadily increasing and the waste treatment plant is currently a net exporter of electricity.
Sludge has yet to be wasted from the reactor in large quantities. Small amounts have been removed for testing purposes; 82 m^3 was removed in the fall of 1989 and land applied in liquid form. Also, in late 1989, approximately 20 tonne, dry weight basis, were removed and dewatered on a rented belt filter press to determine the feasibility of such a means to dewater the sludge. The results were favorable, however, more testing is required to determine proper polymer types and dosages prior to the next dewatering exercise slated for 1990.

Over a 16 month period from January 1989 to April 1990, there was 3 284 000 kg of solids removed in the anaerobic reactor. These solids were made up of raw solids, waste activated sludge, and biomass growth. Over the same period of time, the solids build-up in the reactor was 344 400 kg. Therefore, the solids destruction for this time period was approximately 90%.

Table 4 presents a summary of the performance of the waste treatment plant for the most recent 12 month period. It includes data on the raw waste, BVF effluent and final effluent, COD and SS, as well as overall removals. Table 4 also presents data on the biogas consumed by the gen-sets and the corresponding monthly net power production.

The monthly COD and SS values are based on composite samples collected and analyzed weekly. The volume of biogas burned is recorded daily from a flowmeter located on the gen-set gas train. The net power production is taken from power meters that record import power and export power.

TABLE 4. SUMMARY OF PERFORMANCE
PREVIOUS 12 MONTH PERIOD

	AVG COD (mg/L)				AVG SS (mg/L)					
	RAW WASTE	BVF EFF	FINAL EFF	REMOVAL (%)	RAW WASTE	BVF EFF	FINAL EFF	REMOVAL (%)	BIOGAS BURNED (m3)	NET POWER PRODUCTION (kWh)
1989 JUNE	4340	595	373	91.2	1994	383	130	93.5	149,980	72,000
JULY	3382	594	266	91.5	1592	485	110	92.8	106,700	58,800
AUG	3180	691	119	93.8	1508	462	69	95.3	137,970	61,200
SEPT	2900	296	163	94.4	1530	203	60	96.1	127,930	42,200
OCT	3423	502	310	91.6	2180	380	154	93.7	146,060	8,400
NOV	3744	1324	306	91.1	1784	870	110	93.3	27,740	(122,400)
DEC	4697	540	341	92.7	2173	367	130	93.7	11,240	(124,800)
1990 JAN	3310	395	148	95.6	1673	109	49	97.1	15,410	(56,400)
FEB	4443	351	220	95.0	2125	211	70	96.6	123,630	8,400
MAR	4103	535	206	94.6	2073	370	84	95.8	167,614	84,000
APR	4348	583	178	95.8	2255	403	55	97.6	215,317	92,400
MAY	3832	679	211	94.5	1642	465	80	95.0	147,770	114,000
AVG	3809	590	237	93.5	1877	392	92	95.0		
TOTAL									1,377,361	237,800

Figures 4 and 5 present the COD and SS data, respectively for the previous 12-month period. These figures show that the low loadings to the ASP from the anaerobic reactor have resulted in high overall removals of both COD and SS. Performance of the anaerobic reactor has been enhanced by the addition of a polymer to the raw waste feed in order to improve solids settling within the reactor. The polymer is added at a dosage of approximately 2 mg/L.

The high performance of the plant has enabled it to consistently meet the final effluent limits since early 1989. The limits are based on Federal Guidelines for potato processing plants and are 1.8 kg of BOD/tonne of raw product and 1.6 kg of SS/tonne, on a monthly average. Figure 6 presents the final effluent data from January 1989, to date (May 1990).

Figure 7 presents biogas burned and net power production data from Table 4. As with the performance of the anaerobic and aerobic systems for the plant, the generating capabilities have been improving. The plant is in a net exporting position for the past 29 months, though the dip in production in November and December, 1989 was due to biogas blower controller problems, and in January 1990, it was due to a four-week shutdown of the processing plant. Figure 8 presents the 1990 power production to the end of May. The trend clearly indicates an excellent year is in the works.

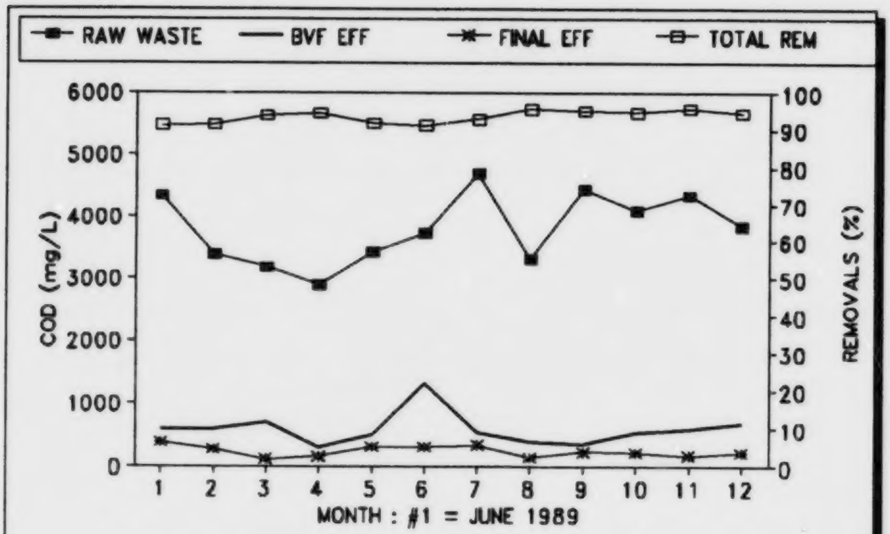


FIGURE 4: COD REMOVAL IN PLANT

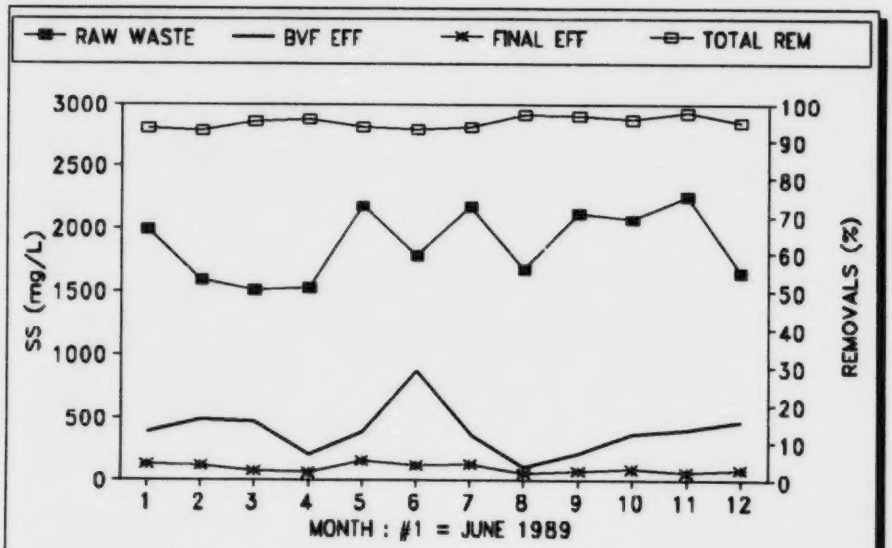


FIGURE 5: SS REMOVAL IN PLANT

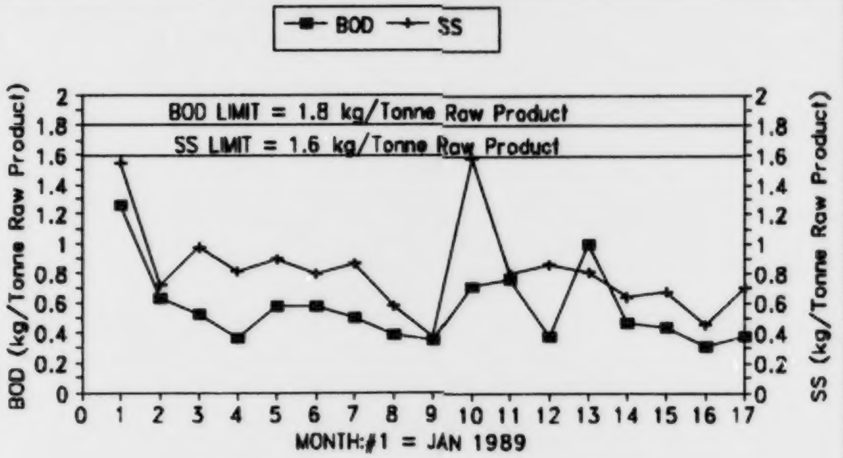


FIGURE 6: FINAL EFFLUENT
MONTHLY AVERAGES

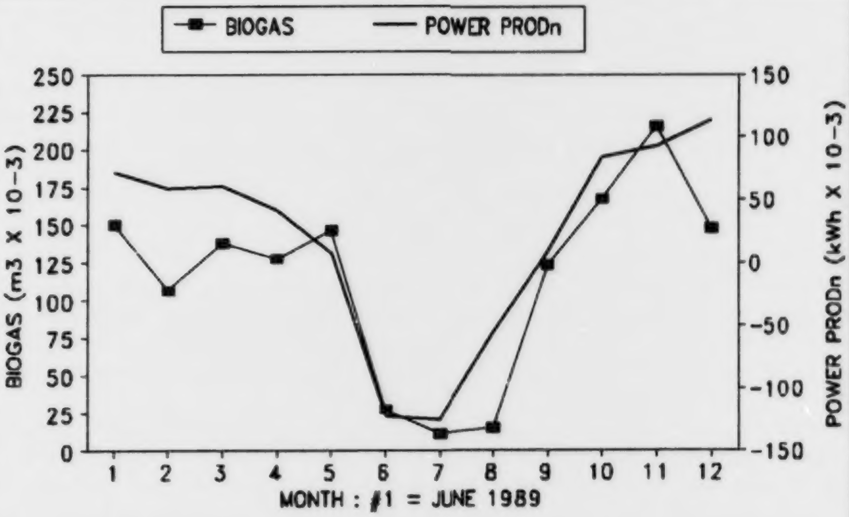


FIGURE 7: BIOGAS & POWER PRODn

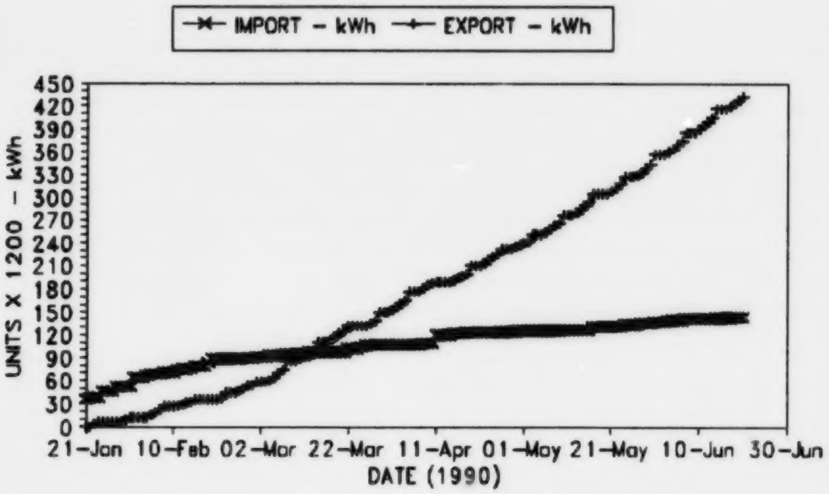
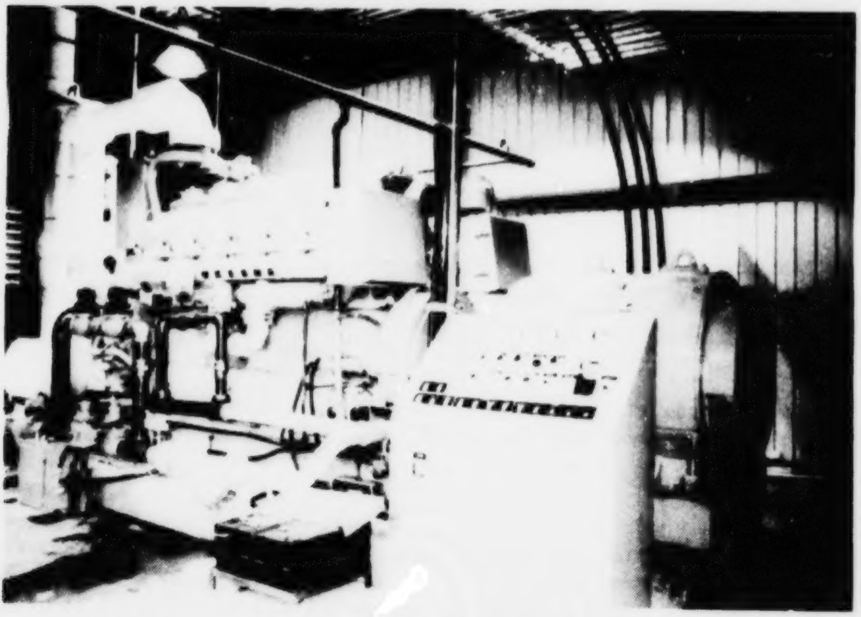


FIGURE 8: POWER PRODn



One of two 600 hp engine gen-sets at the wastewater treatment plant

Conclusions

The anaerobic pilot plant demonstrated that the wastewater produced by this processing plant was treatable to a high degree and the resulting full-scale design was based on 85% removal of COD and SS within the anaerobic unit. Construction of the anaerobic reactor commenced in late 1985 with start-up taking place early in 1986. Due to low quantities of quality seed sludge, the reactor took longer to mature than was originally anticipated, however, once the reactor matured, it was able to achieve design removals. Solids digestion within the reactor are about 90%. Consequently, very little wasting from the system has taken place to date.

The production of biogas and its utilization in the gen-sets has reduced overall operating costs. The system has reached a stage where all electrical requirements are satisfied by the gen-sets and the excess power is exported to the processing plant. As well as producing electrical power for export, the gen-sets aid in maintaining a higher temperature in the digester while providing heat for the control building.

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CONVERSION OF FACULTATIVE LAGOONS
TO MECHANICALLY AERATED LAGOONS

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1.0 Introduction

1.1 The Need

Many small to medium sized communities are seeing a steady growth in population. This, together with an increased per capita rate of sewage generation is, in many cases, overtaxing those communities' sewage treatment facilities.

Public Awareness and social conscience are now demanding the proper protection of our environment. This includes the need to mitigate against overloading water courses with pollutants.

Communities are no longer prepared to accept the nuisance of malodours from poorly operated or overloaded sewage facilities within their neighborhood.

The old adage that there were no votes in sewage is no longer appropriate in today's age of environmental awareness.

All these factors are causing communities and businesses to take a careful and critical look at their sewage treatment facilities with a view to responsible handling of their wastes.

2.0 The Goals To Be Achieved

2.1 The Goals

In its simplest terms, the goal of a sewage treatment facility is to meet the conditions of its license to operate. This will typically require it to produce an effluent of sufficient quality such that its discharge to a receiving water course will have no short or long term detrimental effect on the quality of that water course.

When this goal is not being achieved consistently, the facility's operating license will be called upon by the regulatory authority to take necessary steps to re-establish compliance.

2.2 Changing The Goals

In other cases, the conditions of the license may be made more stringent because of changes in circumstances, e.g. depletion of natural flow in receiving water course;or a significant increase in volume of effluent discharged to a water course. Again, this will require remedial action by the licensee to meet these new conditions.

2.3 The Costs

It is recognized that constructing sewage treatment facilities involves major capital expenditure. It is this fact that has encouraged engineers and owners to evaluate carefully the potential of an existing facility before abandoning it in favour of a new one.

Existing facilities represent a major capital investment, certainly for most small and medium sized communities. By utilizing existing facilities to the maximum it is possible to defer and possibly eliminate the need for major capital investment in new facilities.

It has been shown that existing facilities can be upgraded by conversion to provide a higher quality effluent and/or treat a greater volume of wastewater.

This paper discusses the conversion of facultative lagoons to mechanically aerated lagoons to achieve these very goals.

3.0 Existing Facultative Lagoon Systems

3.1 How They Work

Facultative Lagoons are usually the first system a community will build to handle its wastewater treatment. The typical system comprises:

* anaerobic cells (usually four)
*aerobic pond, and
*storage pond(s)

Facultative treatment is achieved by a combination of anaerobic and aerobic decomposition of biological wastes to a fully stabilized condition. This type of treatment can produce an effluent of very high quality with BOD's below 5 mg/L (compared to the required standard of 20 or 30 mg/L).

3.2 Their Limitations

The process, being a natural one, takes a long time and requires large areas of land for the shallow ponds necessary to achieve oxygen transfer at the surface.

The biological treatment processes are not able to operate effectively in cold conditions and when ponds freeze over in winter the oxygen transfer is essentially eliminated. Consequently, these facilities become effectively little more than storage ponds during winter freeze-up.

Discharge of effluent from facultative lagoon systems is permitted only at certain times of the year when effluent quality is good and/or high dilution factors are available in the receiving water course. Effluent quality is best in the fall, after a full summer's treatment, and dilution factors are best in the spring during run-off. Both spring and fall have, in the past, been acceptable discharge times. However, the common practice now is to discharge once per year for approximately one month in the fall - even though dilution factors can be a problem at this time of year, particularly in small water courses.

This means facultative systems have to store wastewater for the other eleven months of the year. Again, this dictates the need for large areas of land for storage ponds. A typical facility for a population of 10,000 will require in excess of a 1/4 section of land.
Quite obviously, even if suitable land is readily available, a facility of this magnitude is expensive to construct.

Consequently, when facultative lagoon systems become overloaded, the prospect of resolving this by merely expanding the existing system is not necessarily feasible or cost-effective.

Conversion of part of the facility to a mechanically aerated lagoon system should be considered.

4.0 Mechanically Aerated Lagoon Systems

4.1 How They Work

Mechanically aerated lagoon systems accelerate the aerobic treatment processes by introducing and mixing oxygen into the wastewater. In this way, secondary treatment can be achieved within weeks, rather than the months needed in facultative systems.

The most important benefit of this accelerated treatment is the ability to discharge treated effluent continuously, rather than intermittently. This, in turn, eliminates the need for large storage ponds.

A typical mechanically aerated lagoon system will provide for 30 days of aeration, followed by 5 days of quiescent storage, and continuous discharge - provided minimum 10:1 dilution factor can be maintained.

The oxygen is introduced by injecting or mixing air into the water. The mixing is achieved either by mechanical means or the air/water displacement principle.

4.2 Energy And Maintenance Requirements

A major consideration with mechanically aerated lagoons is their relatively high energy requirement, which will typically lie in the range of 20-35 kW per 1000 population. This large range is due to the many factors that effect design such as:

- wastewater strength
- seasonal variations
- oxygen requirements
- treatment system
- pond configuration
- equipment used

Maintenance costs can also be a significant factor in the economic evaluation of aeration systems and equipment selection. Depending on the type of system installed, there will be blowers, aerators, mixers, piping, valving, electrical, control and monitoring equipment to maintain.

It can be important, and in some cases, critical to the operation of aerated lagoon systems to control wastes entering the ponds. Heavy silt loads for example, high strength septic wastes, or illegal dumping of commercial and industrial wastes can all effect not only the immediate performance of the facility but the longevity of the system and the maintenance requirements.

4.3 Operating Variables

Winter operation is the critical time and typically dictates the design criteria. Algae blooms can cause an increase in suspended solids in the effluent during late summer. But when operated and maintained correctly, aerated lagoons can produce a good quality effluent of less than 5 mg/L BOD and can meet year round the typical standard of 20:20.

During summer, when treatment efficiency is at its best, there is the option of cutting back on the amount of aeration energy input which can save on power costs. This must be done with careful consideration of the impact on equipment performance, e.g. blower surging, orifice blocking, and the potential sludge accumulation resulting from reduced mixing.

5.0 The Conversion Process

5.1 General

The conversion process involves modifying:

- lagoon ponds
- control structures
- yard piping

and adding:

- power supply
- aeration equipment
- monitoring and testing facilities
- flow metering, and possibly
- septage/tanker receiving facilities

5.2 Lagoon Modifications

The anaerobic cells of a facultative system will often be too small to consider for conversion to mechanical aeration, in which case they can either:

- be abandoned
- be retained as anaerobic pretreatment ahead of mechanical aeration, or
- be converted to a tanker waste receiving facility.

Abandonment will normally require drain down and infilling, and will be the choice of option where odours are a concern.

Conversion to a tanker waste receiving facility will be appropriate for those facilities that typically receive significant, difficult to handle tanker wastes.

Otherwise, retention of the anaerobic cells as a pretreatment stage prior to mechanical aeration is useful utilization and significantly reduces the solids and BOD loading to be handled by the aeration system.

The aerobic pond in a facultative system will typically be 1.5 metres (5 feet) deep, which is too shallow for economical adaptation to mechanical aeration.

The aerobic pond must therefore either:

- be abandoned
- be deepened to 2.5 - 4.5 metres deep and converted to mechanical aeration, or
- if suitable dilution is not available year round, retained as effluent storage during periods of low base flow in the receiving water course.

Abandoning of the aerobic pond will involve draining down, possibly removing concentrations of sludge to avoid subsequent odour problems, and removing sufficient of the berming to allow natural surface drainage of the area.

If the aerobic pond is of appropriate size for conversion to a mechanically aerated pond, it will need to be deepened because mechanical aeration is not very efficient at depths less than

2.5 m. In fact, static tube type aeration is not typically installed in ponds less than 3.5 metres deep.

The volume, shape and ease with which a pond can be deepened and subdivided into smaller aeration cells must be given careful assessment before establishing its viability for conversion.

The storage pond of a facultative system is usually the best candidate for conversion to mechanical aeration. Storage ponds are typically 2.5 metres deep and more than big enough to accommodate an aeration system that would double the capacity of the original system.

The pond will have to be subdivided into a minimum of two and preferably three cells with a total capacity of 35 days - 30 for aeration and 5 for quiescent settling. The pond width to length ratio should be close to 1:4 to promote plug flow conditions. Excess pond capacity can be used for in-line retention or off-line storage if necessary, or abandoned. If used for in-line retention, algae blooms may become excessive, since the quiescent, oxygen rich effluent thus stored provides an ideal environment for algae growth. This can be controlled by the use of copper sulphate which will kill off the algae.

5.3 Factors to be Considered

There are numerous other considerations to be weighed in the decision to convert:

* the soundness of the existing pond's embankments?
* do the existing ponds leak?
* is the location acceptable for the community's long term development plan?
* is there an odour nuisance at this location?
* are ground conditions conducive to deepening?
* if subsurface aeration equipment is used, the bottom conditions must be capable of supporting this equipment.
* during conversion process, sewage must still be handled and treated.
* clean-up of existing sludge accumulations prior to installation of aeration equipment can be a major undertaking - disposal of this sludge can be expensive.

Several of these factors are considerations in the evaluation of any sewage upgrading. Some are specific to the conversion process, and these are discussed below.

* Leakage Through Existing Embankments

If the existing embankments, or indeed pond bottom are in poor condition or are leaking there are two possible resolutions to this:

1. remedy the leakage by rebuilding or lining areas that are leaking
2. intercept the leakage and return it to the treatment system

A third alternative would be to do nothing, since it could be argued that the conversion process will result in a more compact pond system which must logically reduce the leakage total, and thus in itself will improve the situation. However, this approach is unlikely to be enthusiastically received by the regulatory authorities if the leakage is significant.

Rebuilding or relining areas to mitigate leakage will first require knowledge of existing ground conditions under and around the pond(s). Geotechnical reports, if available, will provide this information, but supplementary site and sub-surface investigation is normally necessary to confirm conditions have not changed.

The logistics and timing of drain-down, clean-up, rebuild or relining; temporary works; the source of suitable fill material for the work and its impermeability will all have to be considered in the overall plan for successful mitigation of leakage from existing ponds.

The alternative to remedying the leakage is to intercept and return any leakage to the treatment system. This can be done with surface or sub-surface interceptors collecting the leakage and routing it to a sump where it can be returned to the treatment system by pumping.

Again, some geotechnical site investigation will normally be required to enable leakage volumes to be assessed and pumping requirements established.

Provided the technical considerations for the peripheral leakage interceptor system can be satisfied, then this method of dealing with leakage will be more economical than the alternative of fixing up leaky embankments and pond bottoms.

* Increasing Pond Depth

There are two options for increasing pond depth:

* raise the embankment, or
* excavate the bottom.

The embankment can be raised on the inside or the outside of the existing. On the inside, will necessitate drain-down of the pond, but will provide a better barrier against leakage. On the outside will be a simpler operation but consideration must be given to addressing potential leakage or slip"plane development at the interface with the existing embankment material.

It may not of course be possible to build on the outside of an existing facility that is tight against the property boundary.

Excavation of the bottom to deepen a pond will likely be a more expensive alternative to raising the embankment. However, constraints such as site limits, poor availability of suitable fill, system hydraulics and other factors may make this the only practical choice.

It is not unusual for lagoon facilities to be located in low lying areas where ground conditions are less than ideal and ground water table is high. Excavating say 1 to 1.5 metres in these conditions can be difficult, expensive and require special wide-pad tracked equipment.

* Temporary Works

Handling and treatment of wastewater flows during conversion will require temporary works and possibly diversion flows. Clearly, if part of a system is to be taken out of service for conversion, the system's overall performance is likely to suffer.

The adverse impact on the final effluent can be minimized by scheduling the works for the summer when treatment efficiency is at its best.

* Sludge Clean-up

Clean-up and disposal of sludge accumulations will be required if a pond is to be converted for bottom-supported aeration systems such as static tube aeration.

Clearly, the quantity of sludge will determine the magnitude of the problem. Estimates of sludge inventory can be made on the basis of published data. However, each facility is unique and estimates should be verified on site by probing the depth in operating ponds and visual inspection when ponds are drained. Typically, areas around inlets or where tanker wastes are dumped will contain significant quantities of settled sludge.

The most economical solution to sludge clean-up and disposal will be on-site, using sludge pumps for transfer, if space is available. Area fill by shallow lagoon or burial by trench would be used. If land is available nearby, land application at a rate of up to 10 tonnes per hectare, dry weight, could be considered.

If off-site disposal is unavoidable, the economics of dewatering prior to transportation must be assessed.

In the conversion process, it is likely that some pond capacity will not be required for the new mechanically aerated system. This spare capacity could be used for storage and elutriation of the sludge.

Whatever sludge disposal option is selected, two factors must be addressed: The potential for odour problems; and the need to meet the regulatory authority's guidelines and standards.

5.4 Selection Of An Aeration System

General

Two basic types of aeration installation are used for lagoon systems:

* surface mixers/agitators
* sub-surface air injection/mixing

The surface mixer/agitator type is not appropriate in cold northern climates because of the potential problems with freeze-up in the splash zone of this type of equipment.

Of the sub-surface type, there are many variations, but all fall into two basic installation categories:

- those supported off the bottom of the pond, e.g. static tube aerators, and
- those supported by buoyancy forces.

Bottom-supported Installations

These have the distinct advantage of being safely below the winter ice level, and not subject to its unpredictable and sometimes destructive forces.

The static tube aeration system is one such system and has a proven track record of ably handling the rigours of our winters in western Canada. The system does require a sound, clean, level base for mounting the units. This will require the pond be drained and cleaned and possibly the bottom properly levelled before the aerators can be installed.

The system operates well in depths of 3.5 metres or more. The aerator units are supplied with air from a blower building via air distribution piping.

Floating Installations

Aeration systems that are supported by buoyant forces can normally be installed without having to drain the pond. These systems typically comprise an electric motor driven mixer and/or aspirator suspended just beneath the water surface from a floating pontoon type arrangement.

In the context of conversion of lagoons to mechanical aeration, these systems have certain distinct benefits over the "bottom supported" systems:

* no need to drain down the pond * can be installed in an operating facility without really disturbing its operation * can operate in a relatively shallow depth of 2.5 metres typical of facultative storage ponds * no need to prepare the pond bottom for installation * no need to remove existing sludge accumulations.

These advantages can translate to significant savings in construction costs. It must be recognized, however, that these systems have equipment located in the ice formation zone. They are subject to the influences of the ice formation, break-up and movement through the course of winter, and have to be designed accordingly. Heat tracing, power failure provisions, moisture protection of electrical equipment and winter access must all be properly addresses for a successful, operational installation.

6.0 The Benefits and Problems

6.1 The Major Benefits

The benefits of converting existing facultative lagoons to aerated lagoons can be summarized as follows.

- use of existing capital investment is maximized
- increased capacity from existing facilities
- more efficient use of land
- more controllable treatment system
- continuous effluent discharge is possible
- capital cost savings

6.2 The Disadvantages

Some of the problems associated with converting lagoons to mechanical aeration are:

- disposal of sludge accumulations
- renovation of leaky embankments
- handling and treating existing flows during the conversion process
- unacceptable location and odour nuisance.

TREATMENT PROCESS FOR NITRATE REMOVAL FROM WATER

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INTRODUCTION

Concerns over the increase in nitrate concentrations in groundwater used for drinking purposes relate to health effects particularly on young children. There is a potential risk to public health because of the possibility of the development of the following two diseases reported to be associated with the consumption of water containing high nitrate levels.

Methaemoglobinemia (Infantile Cyanosis)

This is a disease primarily affecting infants; the acute toxicity of nitrate occurs as a result of its reduction to nitrite, a process that can occur under specific conditions in the stomach and saliva. The nitrite ion formed oxidizes iron in the haemoglobin molecule from ferrous to ferric state. The resultant methaemoglobin is incapable of reversibly binding oxygen, and anoxia or death may occur if the condition is left untreated. Most reported water-related cases of infantile methaemologbinemia have been associated with the use of water containing more than 10 mg/L nitrate-N. Thus, drinking water guidelines and standards recommend that the concentration of nitrate in drinking water should not exceed 10 mg/L nitrate-N.

Carcinogenesis

There has been recent interest in the cancer risk associated with the consumption of drinking water with elevated levels of nitrates. Nitrites (indirectly nitrates) can react with amines and amides to form nitrosamines and nitrosamides. Most N-Nitroso compounds are carcinogenic in a wide range of animal species and most are mutagenic. The epidemiological evidence suggests that high nitrate ingestion may be a contributing factor in gastric cancer. There appears to be little information available to draw any specific conclusions about human cancer and high levels of nitrates in drinking water (Fraser *et al.*, 1980; Glidewell, 1990).

Various physical-chemical and biological processes are available for the removal of nitrate from water. The two types of treatment processes most widely favoured are ion exchange and biological denitrification.

This paper reviews the various processes and includes a case study on an existing ion exchange plant reported in the literature. Costs for other processes are also included in this review.

Biological Denitrification

Biological denitrification is a well established process in the realm of wastewater treatment. However, this process has not been introduced to the field of water treatment on any significant scale. The primary reason behind the slow transfer of technology from the wastewater treatment to potable water treatment is the obvious concern over potential bacterial contamination of the treated water supply.

In biological denitrification, facultative microorganisms are contacted with the water supply containing nitrates and an added carbon source under anoxic conditions [i.e. devoid of molecular oxygen]. Under these conditions, the bacteria utilize nitrates as a terminal electron acceptor in lieu of molecular oxygen. In the process, nitrates are reduced to nitrogen gas which is harmless. The extraneous carbon source is necessary since it supplies the energy required by the microorganisms for respiration and synthesis. If methanol is used as a carbon source, the reaction could be written as follows:

$$6NO_3^- + 5CH_3OH \dashrightarrow N_2 + 5CO_2 + 7H_2 + 6OH^- \qquad [1]$$

The above equation only describes the energy reactions. If bacterial synthesis is considered, the overall denitrification reaction can be written as follows:

$$NO_3^- + 1.08CH_3OH \longrightarrow 0.0625C_5H_7NO_2 + 0.42N_2 + 0.76CO_2 + 1.44H_2O + OH^-$$

When a simple carbon source such as methanol or acetic acid is chosen, the biological solids produces during this process will be correspondingly low; a useful characteristic in that the overall sludge production is minimized. Although nitrates are nearly depleted in the denitrification reactors, the effluent from these columns contains residual suspended as well as dissolved organic solids. The suspended solids are made up of bacterial mass that is dislodged from the reactors by the hydrodynamic shear caused by the upward movement of water and product gas in the reactors. The dissolved solids are presumed to be made up of residual carbon that is not entirely depleted during the reaction. Residual organics remaining in, or imported to the water by the denitrification system, must be removed from the water supply to avoid problems when disinfection by chlorine. When water containing residual organics is chlorinated, halogenated organics may be formed. Some halogenated organics are suspected carcinogens and therefore an alternative disinfection method such as ozonation may be required if residual organics are not completely removed (Dahab, 1988).

Full-scale application of biological denitrification can be considered under several conditions which decrease the opportunity for bacterial contamination of the water supply and provide subsequent treatment required to polish the denitrified effluent to meet drinking water standards (Dahab, 1988). These may include the following cases:

1. Biological denitrification followed by natural storage - Natural storage would provide an opportunity for the water supply to be aerated. The aeration would allow for residual organics to be oxidized by the same biological solids discharges as suspended solids from the denitrification process.

2. Biological denitrification followed by mechanical aeration before treatment- Mechanical aeration will be necessary in large scale applications where natural aeration will not be sufficient to accomplish the same results.

3. Biological denitrification followed by chemical treatment to remove hardness as well as residual suspended solids - This alternative will be useful where the raw water conditions are such that softening using lime and soda ash is required. This sequence is generally followed by filtration on granular media and disinfection.

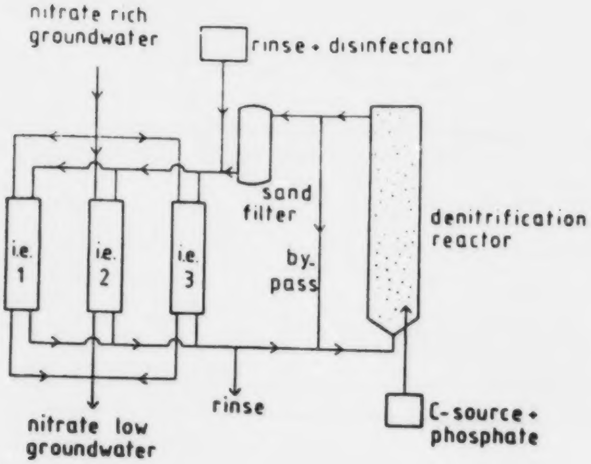
The above cases are presented as examples under which biological denitrification could be introduced and manages as integral unit(s) within the overall scheme of potable water purification. The choice of any particular scheme and the subsequent treatment that might be necessary must be carefully evaluated; this analysis should also include alternative nitrate reduction methods.

Combined Ion Exchange/Biological Denitrification

A new process has been developed [van der Hoek et al., 1988] for nitrate removal from ground water: combined ion exchange/biological denitrification. In this process nitrate is removed from the ground water by ion exchange and regeneration of the nitrate loaded resins is carried out in a closed circuit through a biological denitrification reactor. This reactor removes nitrate from the regenerant so that it can be used again. The process can be operated under different process conditions. Firstly, the regenerant can be varied. Regeneration of anion exchange resins can be achieved with sodium chloride solution or sodium bicarbonate solutions as regenerant. Secondly, it is possible to vary the resin type. Strong base anion exchange resins are normally sulfate selective, but recently some nitrate selective resins have been developed [Guter, 1982] and applied [van der Hoek et al., 1988]. Thirdly, the local ground water composition can vary. Especially the presence of high sulfate concentrations can affect the removal of nitrate from ground water in ion exchange processes; thus sulfate will also affect the combined ion exchange/biological denitrification process. Experiments carried out with a laboratory-scale pilot-plant [van der Hoek et al., 1988] under three difference process conditions illustrate the advantages in using the combined ion exchange/biological denitrification process (Figure 1).

FIGURE 1

Schematic diagram of the laboratory-scale pilot plant
[Adopted from Jan Peter Van Der Hoek et al 1988]



The three process conditions were, treatment of a Dutch ground water with a sulfate selective resin [Duolite A 165] and with a nitrate selective resin [Amberlite IRA 996], both with sodium chloride as regenerant, and treatment of an English ground water containing a high sulfate concentration [181 mg sulfate /l] with a nitrate selective resin [Amberlite IRA 996] and sodium chloride as regenerant. These process conditions are summarized in Table 1. Table 2 shows the pilot plant dimensions, flow rates, raw water composition, regenerant concentration and sludge concentration in the upflow sludge blanket [USB] denitrification reactor during each experimental period. Granular denitrifying sludge [0.5-2.0 mm granules] cultivated on methanol was used. In test C two different ground water flow rates were used: test C1 with a high flow rate and test C2 with a low flow rate.

TABLE 1

Three Process Conditions Tested With The Laboratory-scale Pilot Plant [Adopted from Jan Peter Van Der Hoek et al 1988]

	Test condition		
	A	B	C
Ion exchange resin	Duolite A 165	Amberlite IRA 996	Amberlite IRA 996
Regenerant	NaCl	NaCl	NaCl
Sulfate concentration in ground water	Low	Low	High

TABLE 2

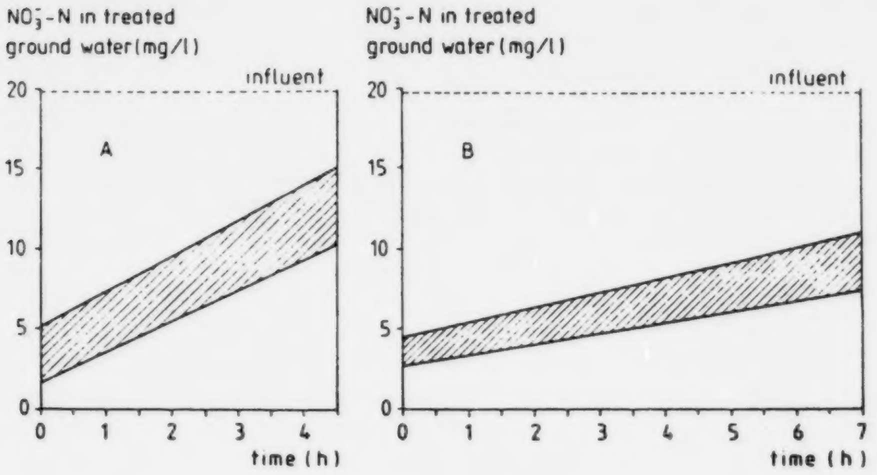
Dimensions of the Laboratory-scale Pilot Plant and Concentrations in Test A,B and C [Adopted from Jan Peter Van Der Hoek et al 1988]

	Test condition		
	A	B	C
Pilot plant dimensions			
volume ion exchange columns (l.)	0.95	1.02	1.45
volume denitrification reactor (l.)	5	5	5
ground water flowrate ($l\,h^{-1}$)	69.2	76.8	71.5/57.4
regeneration flowrate ($l\,h^{-1}$)	9-12	10-12	14-16
rinse flowrate ($l\,h^{-1}$)	9-12	10-12	14-16
Raw water composition			
NO_3^--N ($mg\,l^{-1}$)	19.8 ± 0.7	19.7 ± 0.9	22.7 ± 0.8
SO_4^{2-} ($mg\,l^{-1}$)	31.6 ± 1.8	31.1 ± 1.4	181.1 ± 3.6
Cl^- ($mg\,l^{-1}$)	28.1 ± 1.4	26.2 ± 1.4	91.6 ± 3.5
HCO_3^- ($mg\,l^{-1}$)	104.9 ± 3.8	102.8 ± 4.1	239.3 ± 4.1
pH	7.76 ± 0.15	7.88 ± 0.16	8.20 ± 0.08
Regenerant concentration ($g\,NaCl\,l^{-1}$)	10.1 ± 1.25	10.3 ± 1.25	9.0 ± 1.60
Sludge concentration USB reactor			
suspended solids ($g\,l^{-1}$)	200	190	175
ash content (%)	62	64	53

In the experiments, artificial ground water was used, made up from tap water. Both resins used in the experiments were strong base macro porous anion exchange resins. Duolite A 165 is a sulfate selective resin with a total exchange capacity of 1.19 equiv/L. Amberlite IRA 996 is a nitrate selective resin with a total exchange capacity of 1.01 equiv/L. Figure 2 shows the nitrate concentration in the treated ground water in tests A and B.

FIGURE 2

Nitrate Concentration in the Treated Ground Water During Test A and Test B (Adopted from Jan Peter Van Der Hoek et al 1988)



All measurements relate to the process cycle of 4.5 (test A) or 7h (test B). Table 3 shows the comparison in the sulfate, chloride and bicarbonate concentrations in the treated ground water in both tests A and B.

TABLE 3

Effluent Sulfate, Chloride and Bicarbonate Concentrations in Tests A,B, C1 and C2 (Adopted from Jan Peter Van Der Hoek et al 1988)

Effluent concentration $(mg\,l^{-1})$	Test condition A	B	C1	C2
SO_4^{2-}	0.0–5.7	3.4–27.9	106.1–199.1	99.8–184.0
Cl^-	72.0–108.1	46.7–86.0	98.6–185.8	133.3–194.6
HCO_3	78.1–106.8	98.8–115.9	237.3–261.1	247.1–258.7

While in test A (sulfate selective resin) sulfate is almost completely removed, it is only partly so in test B (nitrate selective resin). Hence the rise of the chloride concentrations in the treated water was moderate in test B as compared with test A, because in test B a part of the sulfate was exchanged for chloride. The difference between the sulfate selective resin and the nitrate selective resin is even more distinct when break-through curves of one single ion exchange column are compared. Figure 3 shows such a break-through curve during the 9h service mode of one column in test A, and Figure 4 shows 14h service mode of one column in test B. In test A nitrate breaks through first, but in test B the situation is opposite; nitrate breaks through after sulfate does. Besides differences in the composition of the treated water, differences in the regenerant composition were also observed. The regenerant was replaced every 6 days in test A and every 11 days in test B. In both cases the pH and alkalinity increased in the regeneration circuit to high values within a very short time as a result of the biological denitrification reaction with methanol. In test A, alkalinity rose to 70-75 meg/L and the pH to 8.9 and in test B to 100 - 110 meg/L and 8.9 respectively.

FIGURE 3

Break-through curve of a Single Ion Exchange Column in Test A (Adopted from Jan Peter Van Der Hoek et al 1988)

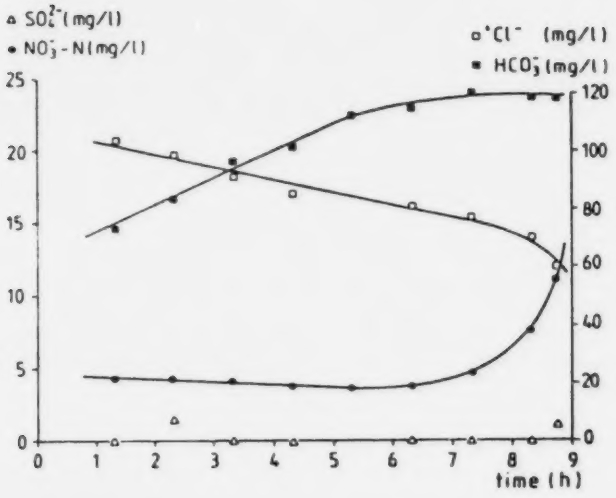


FIGURE 4

Break-through Curve of a single Ion Exchange Column in Test B
[Adopted from Jan Peter Van Der Hoek et al 1988]

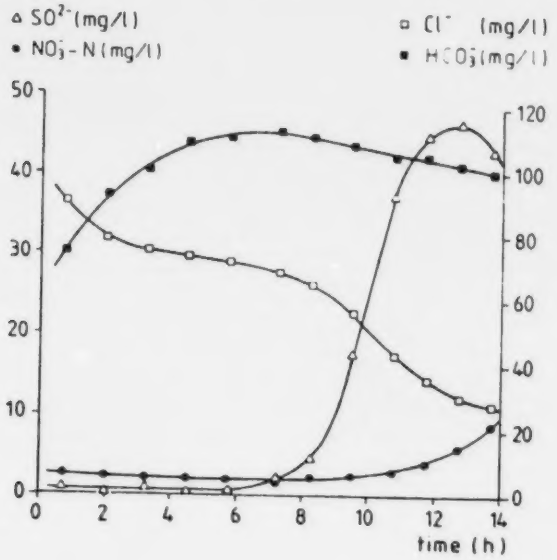
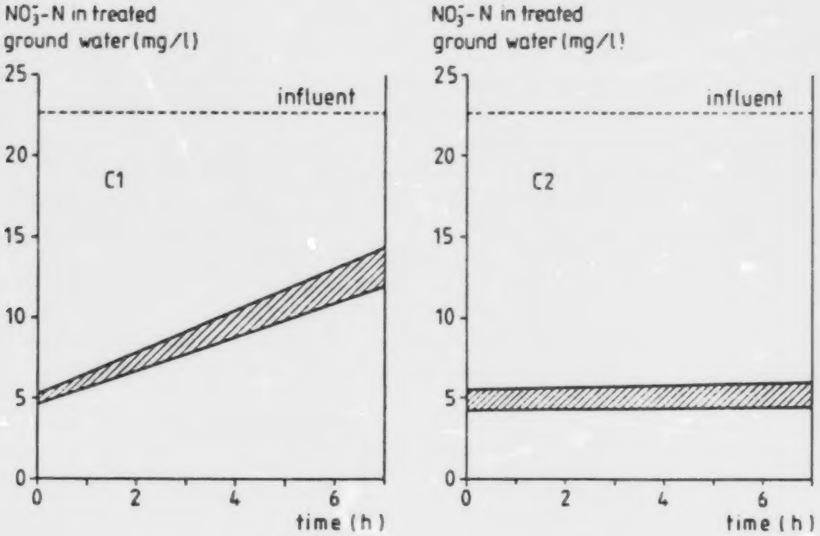


Figure 5 shows the nitrate concentration in the treated ground water in test C. A ground water flow rate of 71.5 L/h, comparable with the flow rates in test A and B, resulted in nitrate concentrations above 11.3 mg/L nitrate - nitrogen at the end of the 7h process cycle [test C1]. When the flow rate was reduced to 57.4 L/h [test C2] it was possible to attain the guide level of 5.6 mg/L nitrate-nitrogen in spite of the high sulfate concentration in the ground water. Compared with test C1, effluent chloride concentrations were higher in test C2 since nitrate removal was better in test C2. The high sulfate concentration in the effluent demonstrates the nitrate selectivity of Amberlite IRA 996. The regenerant in test C2 was replaced every 13-14 days. During these periods the alkalinity rose to 110 - 120 meg/L and the pH to 9.1, thus, as in tests A and B, in this case also, alkalinity played an important role in the regenerant besides chlorides. Thus, the above experimental results showed that for the removal of nitrate from ground water containing low sulfate concentrations, the use of nitrate selective resins offers no advantages over the use of sulfate selective resins in the combined ion exchange/biological denitrification process with respect to effluent nitrate concentrations. However, with the use of the nitrate selective resins, chloride concentrations in the treated water are lower as compared with sulfate selective resins, because sulfate is only partly exchanged for chloride. For removal of nitrate from water containing high chloride concentrations, this can be a benefit. The experiments also showed that water with extremely high sulfate concentrations can also be treated with the combined process when a sulfate selective resin is used. The regenerant was renewed every 2 weeks under one process condition. Compared with conventional ion exchange regeneration [regeneration for 30 min with 5 bed vol/h], this results in a reduction of brine production of 95%.

FIGURE 5

Nitrate Concentration in the Treated Ground Water During Test C1 and C2 (Adopted from Jan Peter Van Der Hoek et al 1988)



CARIX Process

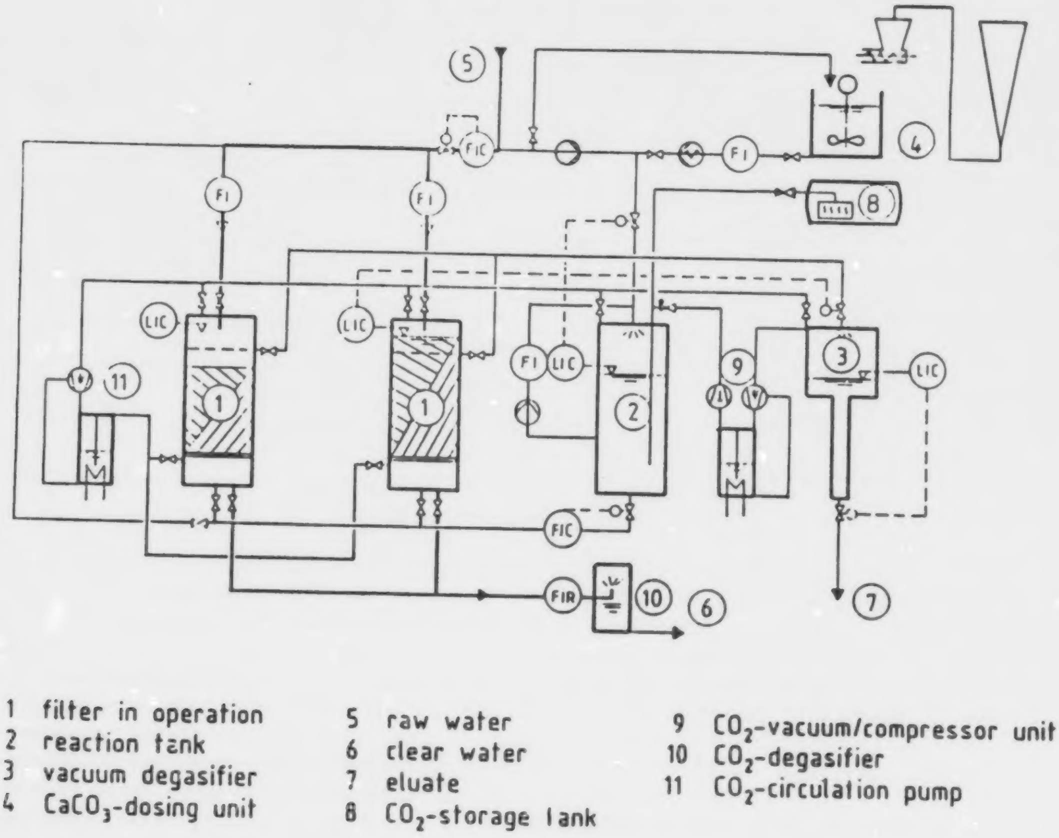
Hagen et al. (1986) presented a method suitable for the preparation of drinking water which was developed during 1972 - 1982 at the Karlsruhe (KfK) Nuclear Research Centre. This method effects the partial softening of water and the removal of nitrate and sulfate with the help of ion exchangers. This method is called the CARIX process.

The CARIX process is based on the combined use of a weakly acidic exchanger in the free acid form and an anion exchanger in the bicarbonate form in a mixed bed. In contact with raw water containing dissolved salts, the exchangers remove neutral salts from solution and liberate equivalent amounts of carbon dioxide. When the exchange resins are exhausted, they can be regenerated by bringing them into contact with a concentrated carbon dioxide solution. In contrast with conventional mixed bed processes, the resins in the CARIX process need not be separately regenerated because each of the mixed resins favorably influences the regeneration of the other. In contrast with conventional exchange processes, the regeneration chemicals do not increase the salt load of the regeneration wastewater. When the concentration of dissolved calcium carbonate and calcium sulfate exceeds saturation, the amount of dissolved salts in the wastewater is less than the amount removed from the raw water. For this reason, this process is particularly environmentally compatible. A substantial property of the CARIX process is the influence of the volumetric ratio of the two exchangers on the type of demineralization. Thus it is possible to vary the ion exchange properties of the mixed bed. When the filter contains a proportion of the anion exchanger, the anions of stronger acids are eliminated together with neutral salts in exchange for bicarbonate ions. When the proportion of cationic exchanger is increased, an additional decarbonization occurs. If the main aim lies in decreasing anion concentration in the water, a mixture with a high proportion of anion exchanger must be used and the regeneration can be carried out in the presence of calcium carbonate in order to increase the pH values.

Due to the fact that the type of demineralization can be influenced in this way, the process can be modified to suit raw waters containing different types and concentrations of salts, or modified to achieve a particular water treatment aim merely by selecting suitable exchangers or by changing the ratio of the two ion exchangers in the mixed bed. The combined neutral salt elimination/decarbonization has a favorable effect on the product water because the ratio of the concentration of the hydrocarbonate to that of anions of stronger acids does not change very much. The CARIX process has been tested in a pilot plant in several waterworks using a mobile experimental plant, and it was found that it can be used for an economical partial demineralization of drinking water. The first centralized large scale plant (Figure 6) was constructed in Bad Rappenau (FRG) and started operation in January 1986 (Hagen et al. 1986)

FIGURE 6

Simplified Scheme of the Large-scale Plant in Bad Rappenau (FRG)
(Adopted from Hagen et al 1986)

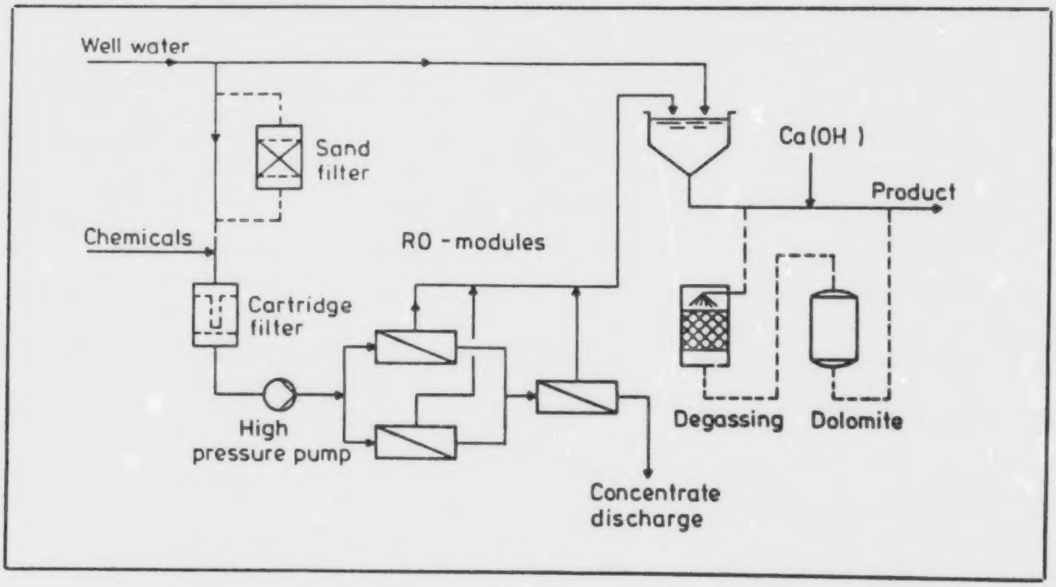

5
FIC
FI
FI
FI
4
8
LIC
LIC
FI
LIC
9
3
LIC
11
1
1
2
FIC
FIR
10
6
7
1 filter in operation
2 reaction tank
3 vacuum degasifier
4 CaCO₃-dosing unit
5 raw water
6 clear water
7 eluate
8 CO₂-storage tank
9 CO₂-vacuum/compressor unit
10 CO₂-degasifier
11 CO₂-circulation pump

Reverse Osmosis

Reverse Osmosis (RO) is a pressure-driven process. At transmembrane pressure differences higher than the corresponding difference of the osmotic pressures, the osmotic flux is reversed - pure water can be seperated from salt solutions. Because of the high rejection of the salt ions, the necessary reduction of the nitrate concentration of well water will be achieved by treating only part of the total amount of water and bypassing the rest (Figure 7) (Rautenbach *et al.*, 1986). RO however cannot reduce nitrate seperately - the process will reduce the concentration of all the dissolved solids. Consequently the hardness of the feed will be reduced. Since the total concentration of dissolved solids is below 600 mg/L in most cases, the resulting osmotic pressure is low even for high recovery rates of about 80%. Consequently, low pressure modules can be used.

FIGURE 7

Flow-diagram of a Reverse Osmosis Plant for Nitrate Separation from Well Water (Adopted from Rautenbach et al 1986)



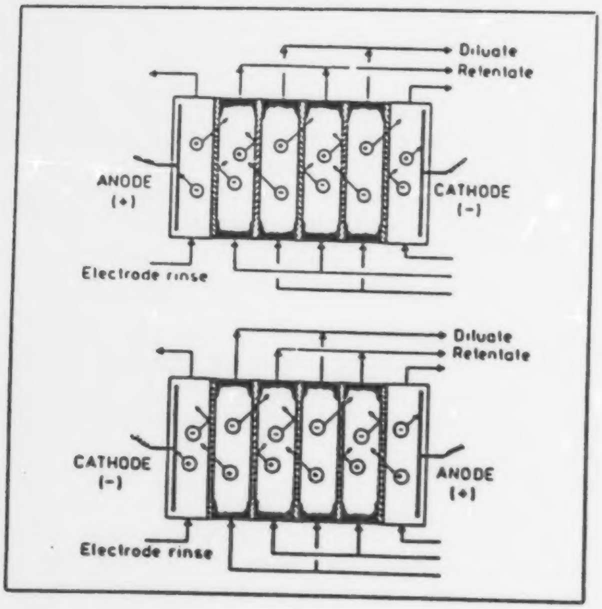
The Institute of Chemical Engineering of the Technical University of Aachen and The Municipal Waterworks of Monchengladbach (FRG) studied the separation of nitrates by membrane processes in a pilot plant. The reverse osmosis unit of the plant had a design capacity of 2 m^3/h permeate, employing spiral - wound modules with FT 30 composite membranes and operating at a feed pressure of about 14 bar. The unit was operated for more than 20,000 h with the first set of membranes. Pretreatment consisted of 5 micro meter cartridge filters only, acid-dosing in order to prevent scaling was proven unnecessary at recovery rates up to 75-80%. Module - cleaning with diluted citric acid was scheduled on a 2-3 months regular basis. The observed nitrate rejection was 93-95%. These experiments led to the conclusion that a nitrate rejection of 90% at a recovery rate of 80% can be expected for large scale plants equipped with FT 30 membranes. In all cases where the concentrate can be discharged without costs and where the water losses are unimportant, recovery rates of RO of about 75% will be optimal. However, in cases where discharge is impossible or costly, the RO-brine has to be further treated. The dissolved solids can be recovered in the solid form by evaporation as a final step. Since the specific costs of evaporation are high, the recovery rate of the preceding membrane stages should be as high as possible. In such a situation, electrodialysis with polarity reversal (EDR = Electrodialysis reversal), shows definite advantages compared to a second reverse osmosis stage.

Electrodialysis Reversal

Electrodialysis employs two different kinds of membranes - anion and cation selective membranes - which are stacked alternatively and form the "diluate" and "retentate" compartments as shown in Figure 8 (Rautenbach et al., 1986). The feed is concentrated in the retentate compartments and depleted in the diluate compartments as a result of the selectively of the membranes. By reversing the polarity every 3h, scaling of the membranes is avoided or even removed.

FIGURE 8

Principal of Electrodialysis Reversal
(Adapted from Rautenbach et al 1986)



Instead of concentrating the RO brine to total dissolved solids of 14000 mg/L (case 1), electrodialysis reversal (EDR) can be employed for the separation of nitrate and, simultaneously for the production of a highly concentrated retentate (case 2). The amount of salt ions which have to be separated from the EDR-feed is about identical in both cases. As a consequence, the power consumption for ion transport across the membranes will also be about identical. Because of the lower diluate concentration in case of direct nitrate separation by EDR, the current density must be lower. Accordingly, larger amounts of ED-membrane surface area have to be installed. Contrary to RO, the electrodialysis plant has to be designed for treatment of the total amount of well water. In case of nitrate concentration reduction from 100 mg/L to 50 mg/l, the specific costs of electrodialysis reversal are about the same as the costs of reverse osmosis but they increase more rapidly for lower product concentrations. While the higher removing rate of EDR is advantageous [Table 4] [Rautenbach et al., 1986], the more complex arrangement of EDR [i.e. three separate fluid circuits] is disadvantageous.

TABLE 4

Comparison of Reverse Osmosis and Electrodialysis Reversal
[Adopted from Rautenbach et al 1986]

	Reverse osmosis	Electrodialysis
Advantages	Simple design Easy membrane exchange Rejection of suspended particles and organic compounds	High recoveries (EDR) Inexpensive pre- and post-treatment
Disadvantages	Scaling by silica is possible Very sensitive against scaling	Separation of ionic components only H_2-forming in the electrode rinse High specific power-consumption for pumping

Ion Exchange

In ion exchange treatment, the contaminated water supply is passed through an exchange resin bed. During contact, nitrate ions in the water are exchanged for similarly charged ions [e.g. chlorides]. This process will continue until the bed exchange capacity is exhausted. The exchange reaction can be depicted by the following reaction:

$$R\text{-}Cl + NaNO_3 \text{-->} R\text{-}NO_3 + NaCl \qquad (3)$$

In the above reaction, "R" represents the resin immobile [i.e. solid] phase. When the resin's exchange capacity is used up, the resin bed must be taken out of service and regenerated. Regeneration restores the bed's exchange capacity to its original state by reversing the reaction and thus forcing the nitrates out of the resin under the action of a concentrated [brine] solution. The regeneration reaction can be depicted as follows:

$$R\text{-}NO_3 + NaCl \text{—>} R\text{-}Cl + NaNO_3$$

After regeneration is completed, the bed is rinsed and normal operation is restored. Nitrate ion, in competition with the other major anions in a water supply, such as sulfate, bicarbonate and chloride, will be removed by strongly basic anion exchange resins. The amount of nitrate removed per given amount of resin will depend mainly upon the sulfate concentration, the total dissolved solids (TDS), the nitrate concentration and the total exchange capacity of the resin. Because the resin has only a limited capacity and in low TDS waters, sulfate is preferred over nitrate on the resin, these factors will determine to a large degree the cost of ion exchange (Evans 1973).

A continuously regenerated ion exchange process, operated with the resins moving through a closed loop, using a strong anion exchange resin to remove nitrates from drinking water was studied by Buelow et al. (1975). This study was undertaken to determine whether or not the nitrate selectivity of a strong anion resin occurs under all circumstances and whether the process would work as well with a highly mineralized water as it does with water containing low concentrations of anions other than nitrate. Based on the data collected during this study, the conclusions were as follows:

1. High concentrations of nitrates in water are not necessarily associated with other mineral characteristics of the water.
2. The anion selectivity of ion exchange resins changes with substantial changes in the ion concentrations in the water.
3. Typical strong-base anion-exchange resins are more selective for sulfates than nitrates under normal conditions.
4. Other anions in water compete with nitrates for ion-exchange sites.
5. Other substances in water interfere with nitrate removal using a strong anion-exchange resin.
    - a. Silica reduces the nitrate removal capacity of the resin but does not replace the nitrate in large amounts as an exchanged ion.
    - b. Iron precipitates reduce nitrate removal, cake the resin, and require special treatment to be removed from the resin.
6. The cost of salt to regenerate the resin used for nitrate removal by ion exchange is a significant part of the cost of the process.

Case Study

The use of ion exchange for nitrate removal is best illustrated by studying the design and working of an existing ion exchange plant. A 1-mgd ion exchange plant was constructed by McFarland (Calif.) Mutual Water Company to remove nitrates from the city's drinking water (Guter and Lauch 1986). McFarland is located in the southern part of the San Joaquin Valley and has a population of approximately 6000 people. The area is extensively agricultural, and nitrogen fertilizers are probably the source of nitrate in the well water. Table 5 shows the composition of the water drawn from McFarland's four wells and shows that three exceed the USEPA's maximum contaminant level (MCL) of 10 mg/L for nitrate as nitrogen. McFarland has been using two wells (1 and 4) with a total capacity of 1400 gpm. Water from well three is prohibited for public use because of high nitrate levels.

TABLE 5

Composition of McFarland Well Water
(Adopted from Guter and Lauch 1986)

Parameter	Well 1 Sampled 5/8/80	Well 2 Sampled 4/9/80	Well 3 Sampled 5/1/80	Well 4 Sampled 4/1/80
Calcium	28	88	156	78
Sodium	50	65	100	72
Bicarbonate	88	102	121	95
Chloride	28	86	94	51
Sulfate	51	105†	310	182
Nitrate-N	6.8	15.1†	22.1	10.6
Total dissolved solids	235	466	827	485
pH units	7.7	7.2	7.3	7.7

*All measurements except pH are in milligrams per litre.
†Analyses on 5/31/78 showed a sulfate level of 261 mg/L and a nitrate level of 17.6 mg/L.

Initial Research

The initial study concluded that the ion exchange process would be best suited as a well-site treatment for nitrate removal at McFarland. Ion exchange can be operated on demand as required by distribution system needs and does not necessitate additional storage and repressurization. Estimated total costs (capital plus operation and maintenance costs) from the initial study were $0.18/1000 gal for the ion exchange versus $1.00/1000 gal for reverse osmosis. Brine disposal costs were not included for either processes. Additional research resulted in the development of a selective anion exchange resin that prefers nitrate over sulfate and regenerant salt reductions through partial regeneration.

Plant Design

A simplified schematic of the plant is shown in Figure 9. The plant is designed to treat 1 mgd, but normal operations will be to treat part of the water and blend it so that the total well production is 1 mgd with a nitrate concentration that is less than 10 mg/l nitrate-nitrogen. The plant consists of three reaction vessels that are 6 ft in diameter and 10 ft high. Each vessel was designed to contain either a 3 or 5 ft bed depth of anion exchange resin. The three vessels were designed to operate in a staggered fashion as shown in Figure 10. For example, when one freshly regenerated vessel begins its service mode, another vessel will be half exhausted, and the third vessel will start regeneration. Hence, two vessels are always in the service mode and the third is in the regeneration or standby mode. Table 6 gives the bed characteristics, target flows and operating conditions for a 3 ft bed. Table 7 shows the number of regenerations per vessel per day for different operational conditions.

FIGURE 9

Simplified Schematic of the Ion Exchange Plant at McFarland, California (Adopted from Guter and Lauch 1986)

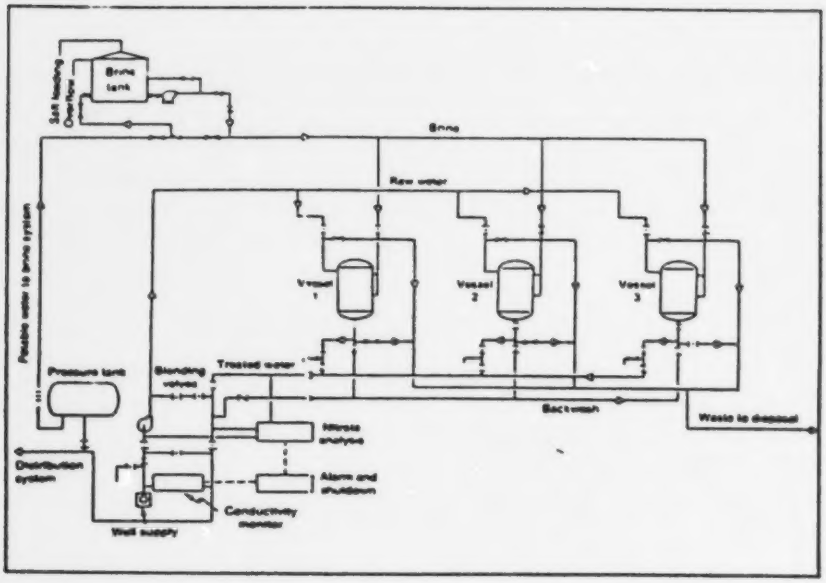

Brine tank
Salt loading
Overflow
Brine
Raw water
Vessel 1
Vessel 2
Vessel 3
Potable water to brine system
Pressure tank
Blending valves
Treated water
Backwash
Waste to disposal
Nitrate analyzer
Distribution system
Alarm and shutdown
Conductivity monitor
Well supply

TABLE 6

Bed Characteristics and Target Flows for a 3 Ft Bed Depth (Adopted from Guter and Lauch 1986)

Bed cross section area	27.7 sq ft (2.6 m^2)
Bed depth	3 ft (0.9 m)
Bed volume	85 cu ft, 635.8 gal (2.4 m^3, 2406 L)
Approximate saturated brine concentration	2.6 lbs/gal (259 g/L)
Saturated brine rate	12 gpm (0.76 L/s)
Brine rinse duration	15 minutes
Diluted brine rate	63.5 gpm (4 L/s)
Bed volume of 6 percent brine at above parameters	1.5
Slow rinse rate	64 gpm (4 L/s)
Slow rinse duration	30 to 50 minutes
Declassification rate	5 gpm/sq ft; 140 gpm (3.4 mm/s; 9 L/s)
Current operating conditions	
Total product water	700 gpm (44 L/s)
Quantity treated	500 gpm (32 L/s)
Quantity treated per vessel	250 gpm (16 L/s)
Bed volume treated per regeneration	250
Salt use (500 gpm [32 L/s], continuous)	2162 lb/day (981 kg/day)
Service loading rate (250 gpm)	9.03 gpm/sq ft (6 mm/s)
Contact time	2.54 minutes
Nitrate-N influent	16 mg/L
Nitrate-N product	6.2-8.3 mg/L

TABLE 7

Frequency of regeneration
[Adopted from Guter and Leach 1988]

Resin Bed Depth and Percent Capacity	Number of Regenerations per Vessel per Day			
	Bed Volume			
	200	250	300	400
100-Percent capacity				
3-ft (0.9-m) Depth	2.62	2.10	1.75	1.31
5-ft (1.5-m) Depth	1.57	1.26	1.05	0.79
70-Percent capacity				
3-ft (0.9-m) Depth	1.83	1.47*	1.23	0.92
5-ft (1.5-m) Depth	1.10	0.88	0.74	0.55
50-Percent capacity				
3-ft (0.9-m) Depth	1.31	1.05	0.88	0.66
5-ft (1.5-m) Depth	0.79	0.63	0.53	0.40

*Current operating condition

The advantages of a 3 ft bed depth are lower initial costs because the vessels would be smaller and a smaller charge of resin would be used. The advantages of a 5 ft bed depth are longer service periods between regeneration and longer resin life because of less frequent regeneration. The initial pilot plant study confirmed that a 3 ft bed depth will give sufficient contact time for the exchange reaction to take place.

Plant Operation

The vessel cycle as shown in Figure 10, which demonstrates that at a 3 ft bed depth each vessel is treating 250 bed volumes in 10.6h. One of the three vessels changes from the service mode into the regeneration mode every 5.3h. This sequence stops when the plant is turned off by the operator or because of low demand. The operational sequence of events for each vessel is (1) service (2) regeneration (3) slow rinse and (4) resin declassification (mixing). After the resin bed is backwashed and expanded in the conventional manner, the resin beads will always settle the same way - the heavier beads will end up at the bottom of the vessel. Therefore, resin declassification (mixing) is necessary when partial regeneration is used, because beads at the effluent end of the vessel would otherwise remain in the nitrate form after regeneration and cause increase nitrate in the product water when the vessel is returned to service.

Figure 11 shows that a considerable reduction in salt usage is accomplished if some nitrate leakage into the treated water can be tolerated. McFarland is operating with a nitrate leakage of 2-5 mg/L nitrate-Nitrogen into the treated water. When this is blended with 30% untreated water (16 mg/L nitrate-nitrogen) the nitrate concentration in the final product water ranges from 6.2 to 8.3 mg/L nitrate-nitrogen. Figure 12 shows an example of increased nitrate concentration at the beginning of service because the resin was not properly declassified.

FIGURE 10

Cyclic Operation of the Three Vessels
(Adapted from Guter and Lauch 1988)

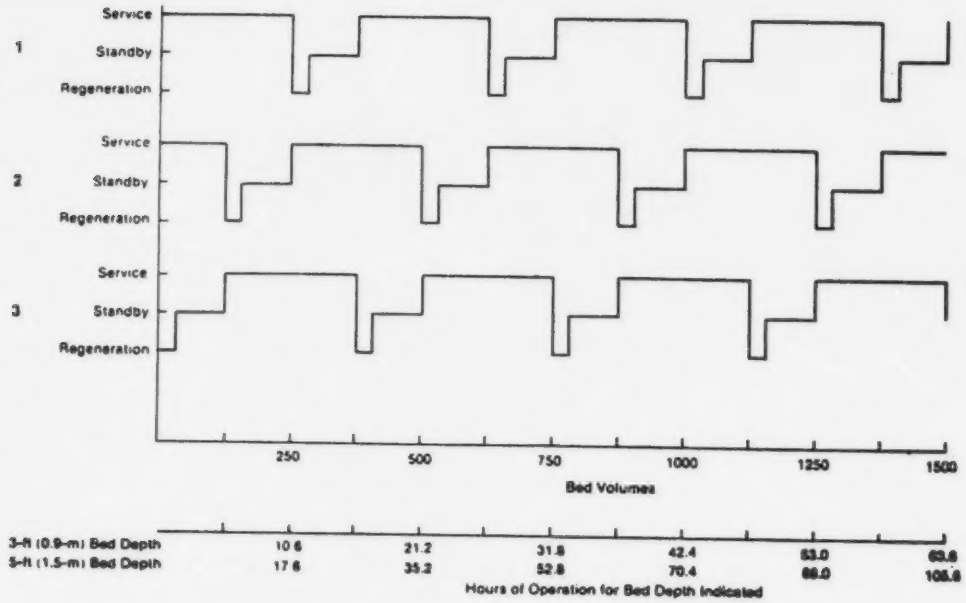


FIGURE 11

Pilot Column Ion Exchange System Nitrate Leakage Dependence
(Adapted from Guter and Lauch 1988)

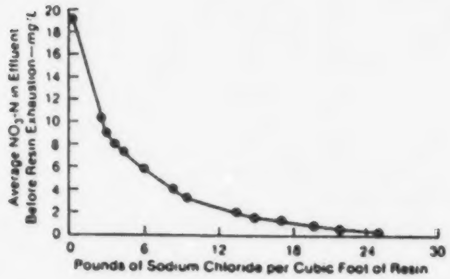
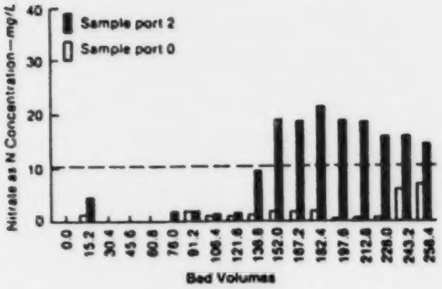


FIGURE 12

Resin Declassification Tests
(Adopted from Guter and Lauch 1986)



Plant Performance

A chemical analysis of untreated, treated, and blended product water is given in Table 8, which shows that nitrate-nitrogen was being reduced from 16 to 2.6 mg/L in the treated water and that the blended product water contained 6.6 mg/L nitrate-nitrogen. Chloride concentration in the blended product water increased from 88.2 to 166 mg/L, the sulfate concentration decreased from 120 to 40 mg/L. The USEPA secondary regulation for both chloride and sulfate concentrations specifies an MCL of 250 mg/L, and Table 8 shows that neither of these ions exceed that level in the blended product water.

TABLE 8

Chemical Analysis of Untreated, Treated, and Blended Water
(Adopted from Guter and Lauch 1986)

Constituent	Untreated†	Treated	Blended
Calcium	94	98	96
Magnesium	2.1	2.1	2.1
Sodium	68	68	68
Potassium	3.8	3.6	3.8
Bicarbonate	119	27.7	120
Chloride	88.2	264	166
Sulfate	120	<5	40
Nitrate-N	16	2.6	6.6
Color units	1	1	1
Odor	none	none	none
pH units	7.2	7.4	7.4
Electrolytic conductivity—*μmho/cm*	790	920	850

*All units are milligrams per litre unless indicated otherwise.
†Iron = <0.05 mg/L, manganese = <0.01 mg/L

Table 9 shows typical data during a cycle for each vessel. The table shows that 1.5 L of 6% brine was used per litre of resin and 5.5 milliequivalents of chloride per milliequivalent of nitrate removed was required for regeneration. The plant would use 2162 lb of salt per day if operated continuously. A typical nitrate breakthrough curve for one vessel is given in Figure 13. The large peak in the middle of the run represents a nitrate calibration concentration of 10 mg/L nitrate-nitrogen. The smaller peaks which are proportional to the calibration peak show that the nitrate-nitrogen concentration of the treated water from the vessel ranged from 4.3 to 4.7 mg/L until nitrate breakthrough.

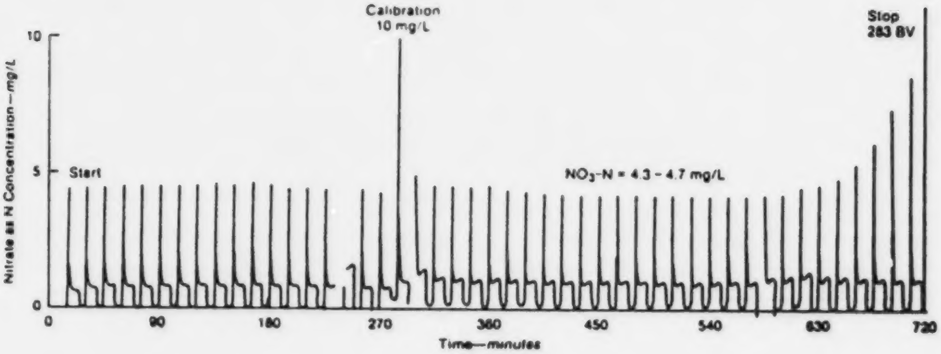
TABLE 9

Operational Data for One Cycle of One Vessel
(Adopted from Suter and Lauch 1986)

Label of Data	Run Data	Bed Volumes	Nitrate-Nitrogen		Calculated Parameter	Calculated Value
			Port 0	Port 2		
Run number	11093-p3	000.0	000.0	000.0		
		0.0	0.0	0.0		
Resin	A 101D	0.0	0.0	0.0	meq NO_3 removed per litre resin	270.07
		15.2	1.4	4.56		
Well	2	30.4	0.0	0.0		
Resin—*gal (L)*	636 (2407)	45.6	0.0	0.0	meq Cl used per litre resin	1499.79
		60.8	0.0	0.0		
		76.0	0.0	1.99		
		91.2	1.99	1.99	meq Cl used per meq NO_3 removed	5.55
		106.4	1.13	1.45		
6% Brine—*gal (L)*	954 (3610)	121.6	0.99	1.58	Bed volume × NO_3 breakthrough	259.4
		136.8	1.4	9.53		
		152.0	1.99	19.08		
Influent NO_3-N—*mg/L*	16.0	167.2	1.9	18.67	meq SO_4	2.50
		187.4	1.99	21.27		
		197.6	0.41	18.79		
		212.8	0.59	18.47	meq NO_3	1.14
Influent SO_4—*mg/L*	120	228.0	0.45	15.69		
		243.2	5.96	15.9	meq S + N	3.64
		258.4	6.95	14.5		
		259.4	10.16	14.5	per litre influent	
Influent Cl—*mg/L*	88.2					
Influent HCO_3—*mg/L*	119					
Number of brine fractions	partial					
Run format					NO_3 removed—*mg/L* (average before breakthrough)	14.33
					NO_3 leakage—*mg/L* (average before breakthrough)	1.67

FIGURE 13

Typical Nitrate Breakthrough Curve
[Adopted from Guter and Lauch 1988]



Plant Costs

Estimated operational and maintenance costs are given in Table 10. Resin replacement and salt costs are the largest constituents of the operation and maintenance costs. The total O and M costs from Table 9 were 12.8 cents/1000 gal. Capital costs were 11.4 cents/1000 gal when amortized at 10% over 20 yrs (table 9). Therefore, the total cost for this system was 24.2 cents/1000 gal.

TABLE 10

Operation and Maintenance Costs
[Adopted from Guter and Lauch]

Item	Annual Costs—$	$/1000 gal
Operator—*1 h/day*	4 745	0.013
Normal O&M—2% ($354 638)	7 113	0.019
Power, booster pump—0.12 hp·h (0.093/kW·h)	8 140	0.022
Resin replacement—5 years	11 522	0.032
Salt—($31.50/ton), 70 percent capacity	12 429	0.034
Miscellaneous	3 000	0.008
Total O&M costs	46 949	0.128

Process Cost Comparison

A Comparison of the approximate costs for nitrate removal by ion exchange, reverse osmosis, and electrodialysis shows that the single-bed chloride-form ion exchange is the cheapest process. However, for nitrate removal from water containing more than 1000 ppm TDS, the membrane processes may be the only reasonable alternatives. In this case electrodialysis would tend to be preferred over reverse osmosis. The rough cost estimates include amortized capital, labour, chemicals, resin and membrane replacement, and energy. Table 11 gives the approximate cost comparison for nitrate removal by ion exchange, electrodialysis, and reverse osmosis. [Clifford and Lin, 1983. Nitrate removal using biological denitrification costs about $0.65/1000 gal of water treated based on the assumptions that the water supply has an influent nitrate-nitrogen concentration of 50 mg/l, treatment efficiency of 90% and a plant size of one million gallons daily [Dahab, 1988]. The existing ion exchange plant for nitrate removal at McFarland [Calif.] has a process cost of $0.24/1000 gal of water treated, which goes to show that ion exchange is the cheapest available process for nitrate removal at present.

TABLE 11

Approximate Costs for Nitrate Removal by Ion Exchange, Reverse Osmosis, and Electrodialysis [Adopted from Clifford and Lin 1983]

Process	$\frac{\$}{1000\ \text{gal}}$	Product Water TDS, ppm
Single-Bed Chloride-Form Ion Exchange	0.30	500
Electrodialysis	0.85	185
Reverse osmosis (Aramid Membrane)	1.00	185

Assumptions: Feedwater = 500 ppm TDS, 21 ppm NO_3-N, 50 ppm SO_4^{2-}
Bypass flow = 30% of blended product flow
Desalting process = 90% overall TDS rejection
NO_3-N = 7.0 ppm in blended product

Conclusions

Based on the preceding discussion of various physical-chemical and biological processes for the removal of nitrate from ground water, the conclusions are as follows:

1. Biological denitrification appears to be a feasible alternative for nitrate removal when the nitrate contamination is quite severe. But a more extensive use of this process is limited by the extreme process control required for this process to prevent bacterial breakthrough and also the additional treatment required to remove suspended and dissolved organic solids imparted by this process to a water supply that is essentially free of organics.

2. Ion exchange proves to be a viable alternative from an economic and practical point of view. The process is well established in water technology and is easily controllable. Among the existing processes, ion exchange has the lowest cost per 1000 gal of water treated, but the spent regenerant disposal poses problems.

3. Membrane processes such as reverse osmosis and electrodialysis are much more expensive than ion exchange and produce a product water with an unnecessarily low total dissolved solids. However, reverse osmosis and electrodialysis may be the only reasonable alternatives for waters containing a high total dissolved solids (TDS > 1000 ppm).

4. combine ion exchange/biological denitrification with its advantage of a significant reduction in brine production offers another alternative for nitrate removal. But whether this process is economically viable on a large scale is difficult to judge since process cost estimates are not available for comparison with other existing nitrate removal processes.

5. The CARIX process may be used for a partial demineralization of water. This process has its advantages in that the process can be modified to suit the raw water quality. Though this process has been used in a large scale plant in the Federal Republic of Germany, a more widespread use has not been indicated in present literature.

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SEWER UPGRADING IN BUILD-UP RESIDENTIAL AREAS

LAGO LINDO SUBDIVISION, EDMONTON, ALBERTA

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Environmental Services
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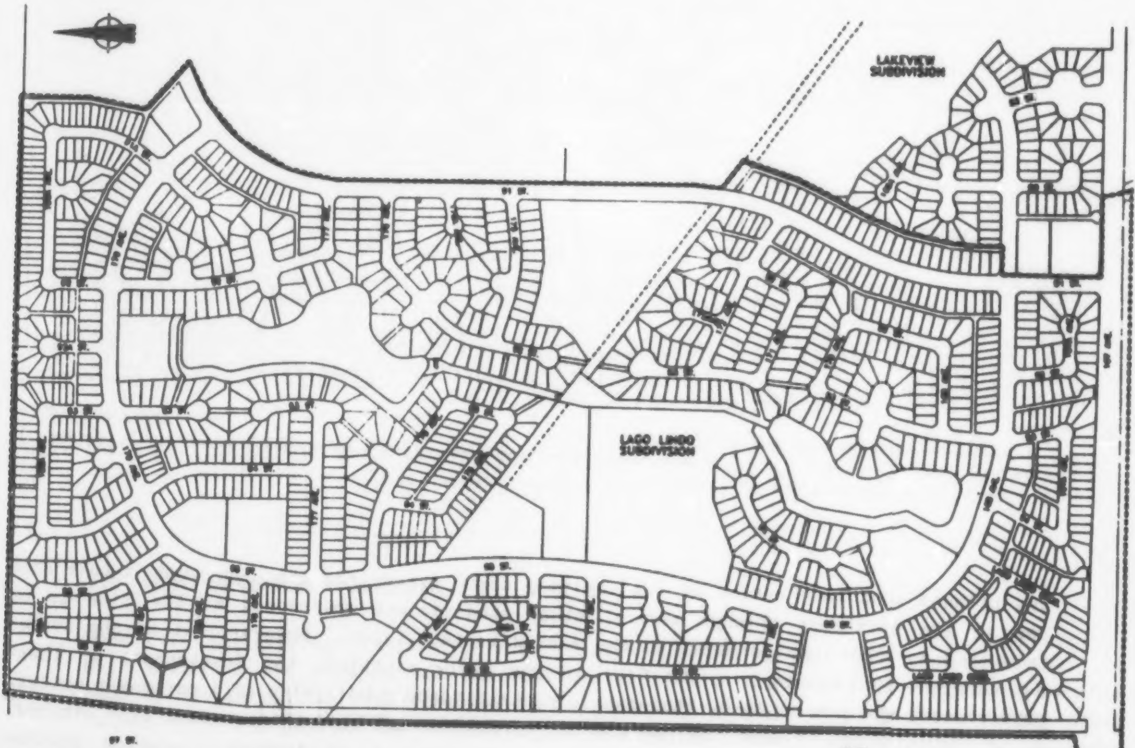
1.0 BACKGROUND AND DETERMINATION OF PROBLEM

1.1 Study Area

The Lago Lindo subdivision is situated in the northeast part of Edmonton, located east of 97 Street and north of 167 Avenue. It is comprised mainly of single family residential units with some row housing, one existing and one proposed neighborhood commercial development. Separate and public schools are also proposed for this subdivision. The area of the subdivision shown on Figure 1 is approximately 124 hectares (306 acres).

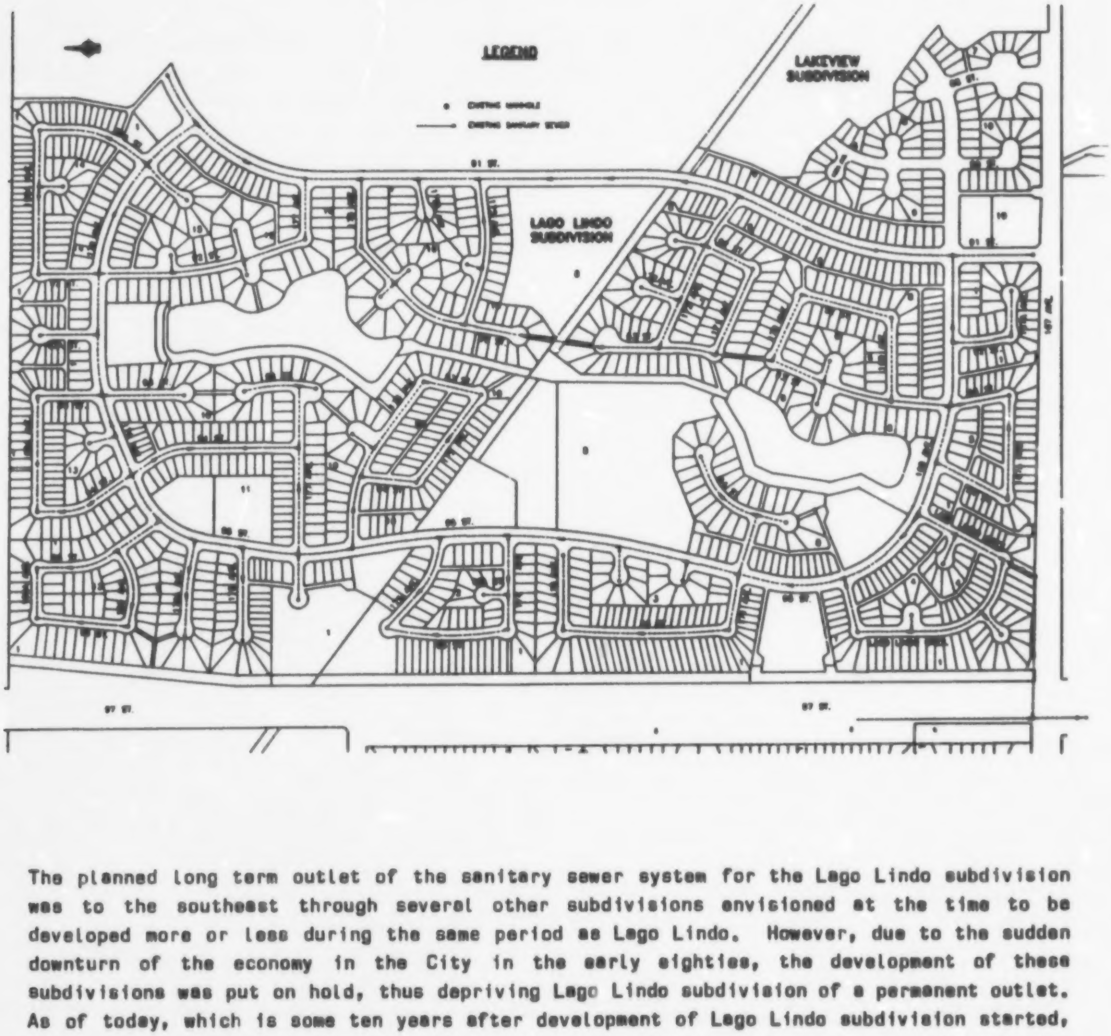
FIGURE 1
Plan of Lago Lindo Subdivision



Development of Lago Lindo subdivision, which began in 1980, occurred in several stages according to the demands for serviced lots in that area. Installation of utilities (water, sanitary and storm sewers and franchised utilities) followed site grading of each subdivision phase.

Construction of the first phase of the sanitary sewer system was started in 1981 and extension of the system took place as the various stages of the subdivision took shape. The existing sanitary sewer system is shown in Figure 2. The sanitary sewer system currently services approximately 1325 single family lots and 164 multi-family units, with a population of approximately 5,000 persons. Practically all of the single family lots have weeping tiles which are connected to the sanitary sewer system.

FIGURE 2
Existing Sanitary Sewer System

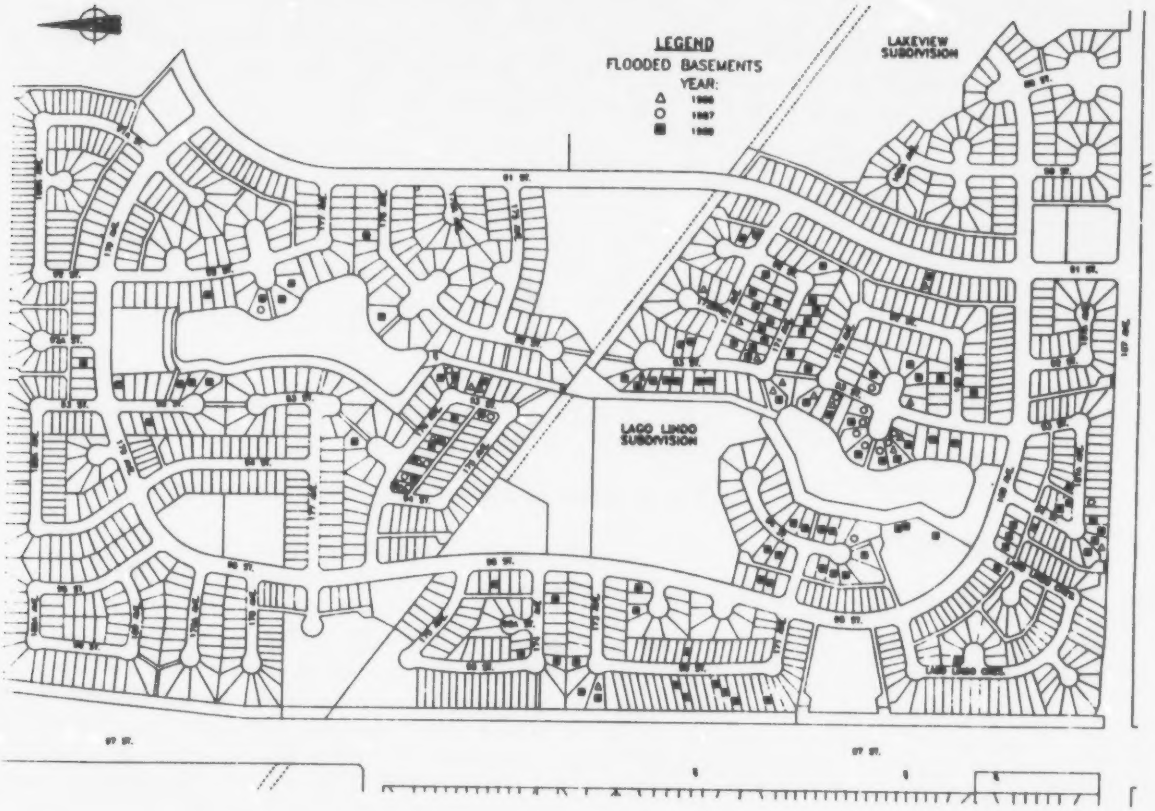


The planned long term outlet of the sanitary sewer system for the Lago Lindo subdivision was to the southeast through several other subdivisions envisioned at the time to be developed more or less during the same period as Lago Lindo. However, due to the sudden downturn of the economy in the City in the early eighties, the development of these subdivisions was put on hold, thus depriving Lago Lindo subdivision of a permanent outlet. As of today, which is some ten years after development of Lago Lindo subdivision started, it's sanitary sewer system is still only connected to a temporary pumping station discharging to a sanitary trunk along 97 Street to the south.

1.2 Background

Extensive basement flooding occurred during the summers of 1986 and 1987. Figure 3 shows the location where basement flooding was reported to the City of Edmonton during these years. Preliminary investigations pointed to sanitary sewer surcharging as the primary cause of basement flooding. In September, 1987 the City of Edmonton retained CH2M Hill Engineering Ltd. to investigate the causes of the recurring basement flooding problem. CH2M Hill started the investigation in November, 1987; but halfway through the exercise, another major storm hit the area in July, 1988 and caused extensive basement flooding again.

FIGURE 3
Locations of Reported Basement Flooding



1.3 Confirmation of Problems

CH2M Hill Engineering Limited, in their final report of April, 1989 confirmed that the main cause of the problem was the overloading of the sanitary sewers due to excessive amounts of inflow and infiltration [I/I] primarily through weeping tiles during heavy rainstorms. The problem was further aggravated by the limited capacities of the temporary pumping station and its downstream sanitary trunk at 97 Street which could barely handle peak dry weather flows.

In order to assess the magnitude of the problem, a theoretical computation of the amount of infiltration through the soil under a 1 in 50 year design storm event using Darcy's Law was carried out. A peak wet weather flow of 0.6 cubic metres per second for the whole subdivision was obtained This figure was close to 4 times the designed peak dry weather flow and was about 7 times the existing capacity of the temporary pumping station.

1.4 Consideration of Alternatives

Since the main problem was the excessive I/I in the sanitary sewers during wet weather, the most logical and effective solution would be to prevent this from happening The main sources of the I/I were found to be the weeping tiles around the houses and manhole covers located at road sags. The latter could be easily rectified by sealing the manhole covers and barrels. However, the reduction or elimination of I/I through weeping tiles would require lot regrading or the reconnection of weeping tiles to sump pumps discharging to ground or to storm sewers. All these methods would result in extensive disruptions of a large number of private properties and were not acceptable to the residents In view of this objection the more expensive alternative of upsizing the existing sanitary sewers throughout the subdivision to handle the extraneous flows was adopted.

With regard to the limitation in capacities of the temporary pumping station and the downstream sanitary trunk, several viable alternatives were considered. These included construction of a new trunk all the way to the wastewater treatment plant or the provision of a storage facility to equalize the flow during wet weather. Different types of storage facilities such as open lagoons, covered lagoons and underground tanks were evaluated After discussions with the residents and maintenance staff, the use of an underground tank based on the concept of gravity flow in - pump out was recommended.

An upgrading program with an estimated cost of $17.23 million was finally approved by the City Council in January 1989 and Associated Engineering Alberta Ltd. was subsequently appointed by the City to carry out the detailed design and construction engineering services for the project.

2.0 SYSTEM ANALYSIS

2.1 Development of Design Criteria and Standards

Based on the preliminary analysis that had been carried out during the investigation phase the City established certain basic design criteria to be used in the analysis and design of proposed improvements. In addition to the basic dry weather flow design factors, point hydrographs were developed to simulate wet weather inflows. Some of the downstream components were oversized to serve other outside contributing areas deemed to be within the service envelope. Table 1 provides a summary of criteria used in the development of design flow hydrographs.

2.2 Dry Weather Flow (DWF)

Dry weather flows including domestic sewage and infiltration were based on the recorded hourly variations and with the following peaking factors:

- 3.3 for the 50-year, 3-hour storm event (for sizing pipes).
- 2.5 for the 50-year, 24-hour storm event (for sizing storage).

TABLE 1

Summary of Design Criteria

Domestic Sewage Flow

.1 Per Capita Average Daily Sewage Contribution: 350 litres.
.2 Peak Hourly Factor: Harman's Formula
.3 Peak Hourly Sewage Flow = Average Daily Sewage Flow x Peak Factor

Infiltration

.1 Pipe Infiltration:
  - PVC Pipe: 5L/day/mm dia./km
  - Concrete and VCT Pipes: 20L/day/mm dia./km
.2 Manhole infiltration (for all manholes): 0.05L/sec/MH
.3 Foundation Drain Infiltration: 0 06L/sec/house
.4 Infiltration Peaking Factors 1.0

Extraneous Flows

.1 Manhole Cover Inflows at Sag Locations
  - 0.6 L/s/MH for 50-year, 3-hour storm
  - 0.3 L/s/MH for 50-year, 24 hour storm

2.3 Wet Weather Flow (WWF)

The Wet Weather Flow was calculated by adding the Dry Weather Flow components noted in 2.2 to the Extraneous Flow components. The time to peak of the extraneous flow hydrographs were matched with the time to peak of the Dry Weather Flow components.

2.4 Restrictions in Downstream Conveyance System Capacity

The 97th Street Trunk Sewer which conveys sewage from the Lago Lindo Lift Station to the City's combined sewage system was identified as having restricted capacity under various flow conditions. The capacity of this trunk, which serves areas other than Lago Lindo, therefore placed limits on the discharge rate contributed from Lago Lindo. The following limits on allowable discharge rate for the new Lago Lindo lift station were established by the City:

- Dry Weather Flow Condition	127 4 L/s (4.5 cfs).
- Wet Weather Flow Condition:	99.1 L/s (3.5 cfs).
- Maximum Discharge Condition during emptying of the Storage Tank:	198 2 L/s (7 0 cfs).

2.5 Analysis of Existing System

The objectives of the evaluation of the existing system were:

.1 To determine if the existing system was adequate to handle the dry weather flows.
.2 To determine if the system was adequate to handle the wet weather flows during 3-hour 50-year and 24-hour, 50-year storm events.
.3 To confirm which components of the system were undersized to carry the peak wet weather flows.

To meet the above objectives and to handle the numerous input data based on the design criteria and the 170 input hydrographs, the EPA SWMM version 3.3 computer model was selected for the hydraulic system analysis. The computer simulation confirmed that the existing sanitary sewer system was inadequate to handle both the dry weather flows and the wet weather flows. It also confirmed the locations of undersized sewer sections leading to the proposed rehabilitation program.

2.6 Analysis of Proposed System

The objectives of the analysis of the proposed sanitary sewer system were:

.1 To develop basic relief alternatives for the overall system.
.2 To establish the required pipe sizes for the proposed relief or replacement pipe sections under:
  a. Free flow conditions
  b. Allowable surcharge conditions (1 metre below lowest basement elevation)
.3 To evaluate the performance of the proposed lift station and storage facilities.

Preliminary evaluation of basic relief alternatives resulted in the conclusion that there were no quick fix or shortcut alternatives which would bypass the overloaded sections of the system. To meet the second objective in the combined model the overloaded pipe sections were replaced with larger pipe sizes determined previously by the TRANSPORT BLOCK simulation of the east and the west sanitary trunk sewer systems. The proposed sanitary sewer system was then modified by the combination of reducing the proposed trunk sewer size of some sections and eliminating or deferring the upgrading of other sections to determine its performance under the allowable surcharge conditions.

3.0 RECOMMENDED SYSTEM IMPROVEMENTS

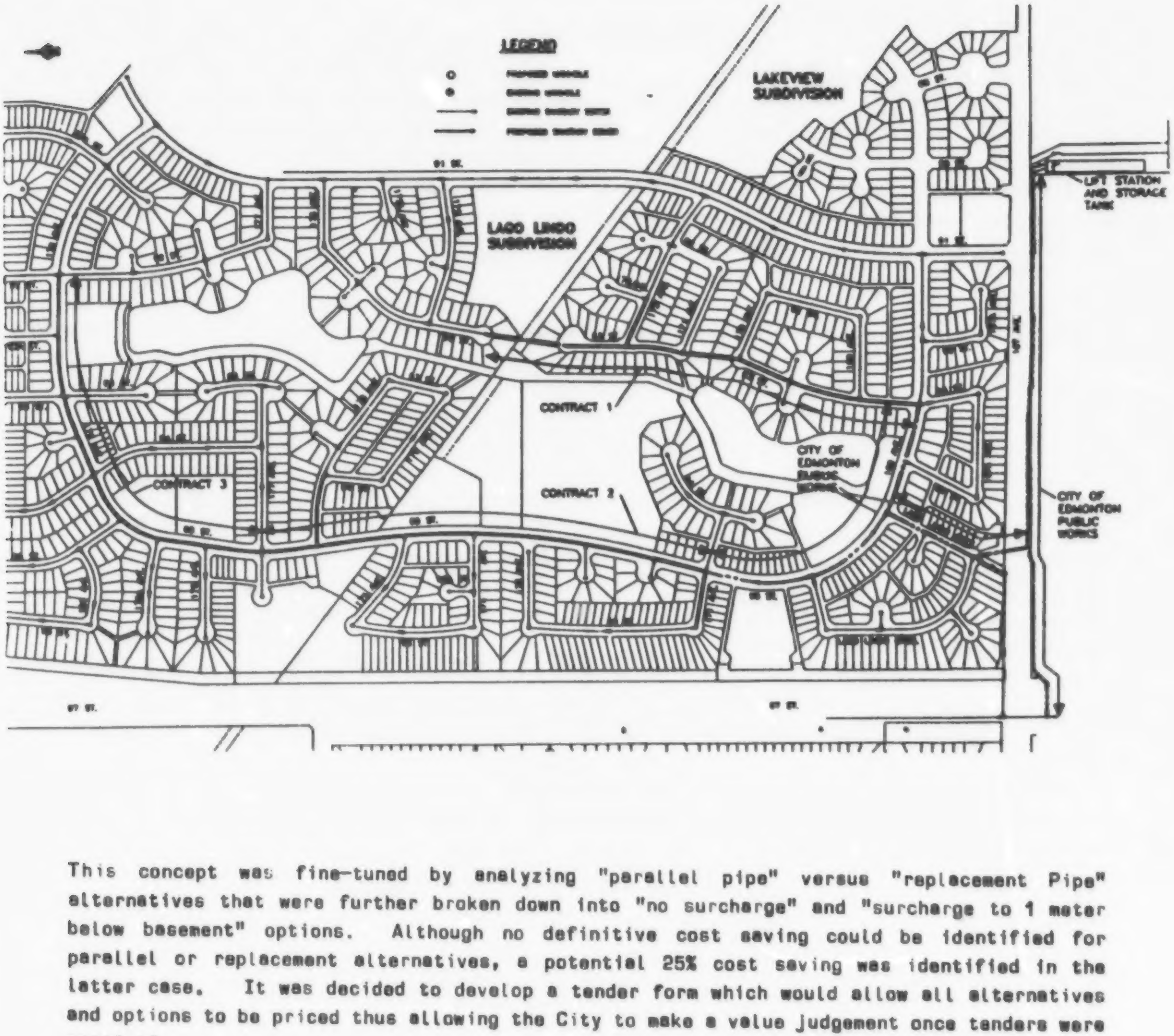
3.1 Gravity Sewers

From the system analysis it had been determined which areas in the subdivision could expect flooding and thus which components of the system required upgrading or relief. In order to analyze various rehabilitation alternatives in a timely and cost effective manner, GIS software was used to allow graphical system analysis A 3-dimensional model of the system was created using AUTOCAD and GEO/SQL - MunMap GIS software. The latter was used to facilitate importing and graphically analyzing data collected from the hydraulic computer analyses. Basically after each system alternative was modelled, profiles of any pipe or group of pipes in the system could be generated showing hydraulic gradients resulting from the proposed improvements related to pipe manhole and street profiles as well as a profile of the existing basement elevations.

Early in the analysis phase it became apparent that there would not be any quick fix short cut gravity sewer relief solution. Because of the extent of development in the area and the layout of the system we concluded that the rehabilitation would consist of upgrading the capacity of existing trunks. This allowed the team to focus in on one basic rehabilitation concept as shown on Figure 4.

FIGURE 4

Proposed Sanitary Sewer Upgrading


LEGEND
LAKEVIEW SUBDIVISION
LAGO LINDO SUBDIVISION
LIFT STATION AND STORAGE TANK
CONTRACT 1
CONTRACT 2
CONTRACT 3
CITY OF EDMONTON PUBLIC WORKS
CITY OF EDMONTON PUBLIC WORKS
91 ST.
97 ST.

This concept was fine-tuned by analyzing "parallel pipe" versus "replacement Pipe" alternatives that were further broken down into "no surcharge" and "surcharge to 1 meter below basement" options. Although no definitive cost saving could be identified for parallel or replacement alternatives, a potential 25% cost saving was identified in the latter case. It was decided to develop a tender form which would allow all alternatives and options to be priced thus allowing the City to make a value judgement once tenders were received.

The final recommended alternative consisted of upgrading of between 2 3 and 3.1 kilometers of internal gravity sewers for tendering by the private sector in addition to 2 kilometers of downstream gravity sewers for construction by City forces.

3.2 Lift Station and Storage Tank

Once the pre-design of the gravity sewer rehabilitation was advanced to the point where inflow hydrographs under both dry weather and wet weather design conditions were known the pre-design of the lift station and storage tank was undertaken. Based on the wet weather inflow hydrograph it was determined that a storage capacity of 11,130 cubic metres would be required.

FIGURE 5
Plan Profile of Lift Station and Storage Tank

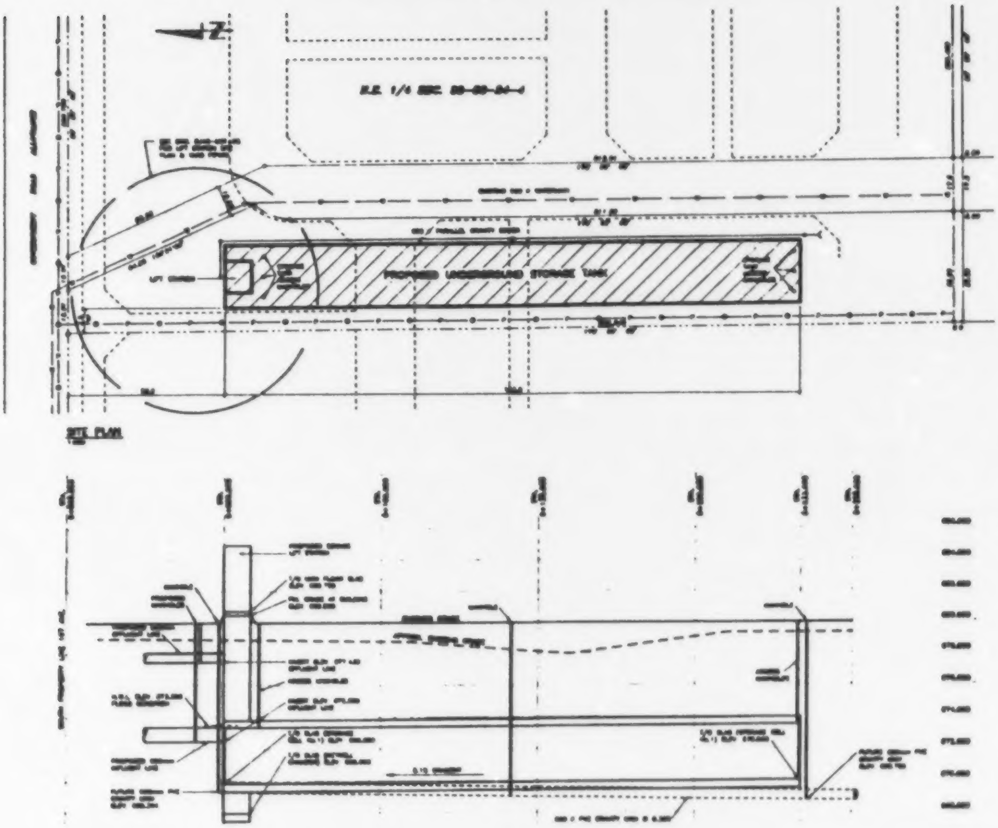
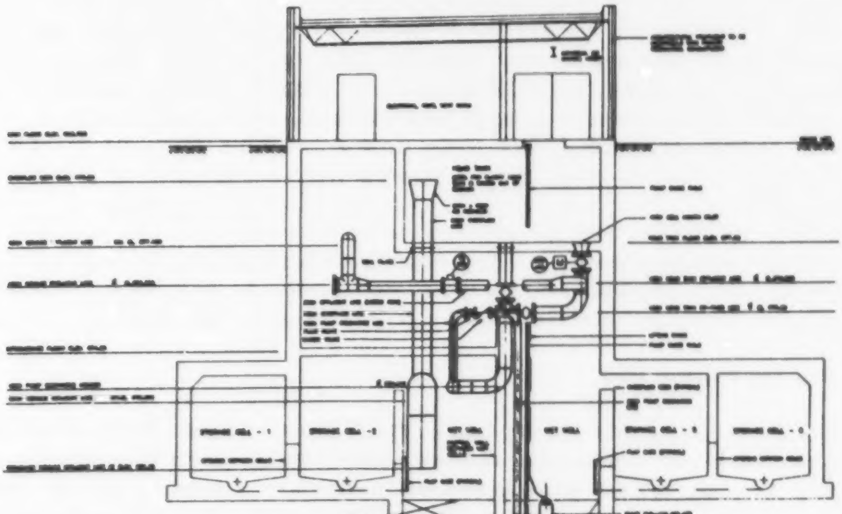

N
N.E. 1/4 SEC.
LIFT STATION
PROPOSED UNDERGROUND STORAGE TANK
SITE PLAN

FIGURE 6
Section Through Lift Station


STORAGE CELL - 1
STORAGE CELL - 2
WET WELL
WET WELL
STORAGE CELL - 3
STORAGE CELL - 4

Primary control of pump start/stop during normal dry weather would be controlled by wet well level. Because of the variable nature of the downstream system and its ability to accept flows from Lago Lindo during and after wet weather events, it was determined that the lift station discharge rate would have to be controlled by flow level in the downstream 97th Street trunk sewer. At the same time we wanted to make maximum use of the 97th Street trunk's available capacity to minimize the required storage tank capacity as well as storage duration periods. It was apparent that the pumping system had to have infinitely variable flow capacity within the established range of flow. Several different pumping concepts were analyzed including the obvious alternative of variable speed pumps. Due to City maintenance staff concerns about complexity and maintenance of variable speed pumps, a less complex alternative consisting of fixed speed pumps, head tank, and modulating flow control valve was adopted. It was recommended that a flow level control manhole be established on the 97th Street Trunk Sewer to remotely control the flow generated from the Lago Lindo Lift Station. This would ensure that the system would be responsive to the hydraulic influence of outside inflows to the 97th Street Trunk Sewer.

The storage tank pre-design presented its own unique set of problems. Because of its depth below ground, a structural concept was required that would cost effectively support the high earth loading on the structure while being flexible to effects of rebound in the supporting shale. At the same time a geometric configuration had to be chosen which would limit the amount and shape of the site required for the facility. Of course there were also the concerns of maintenance and cleaning of the facility after each wet weather event. Several configurations were analyzed however the recommended configuration consisted of a series of 6 parallel tubes gently sloping up from the lift station wet well to allow self draining and incorporating washdown piping for cleaning proposes. By using the parallel tubes we were able to take advantage of the natural structural efficiency of the tubular shape at the same time as allowing the tank to fill in stages to minimize cleaning requirements after each wet weather event. We were also able to take advantage of a long and narrow strip of land available for siting the facility immediately south of Lago Lindo.

4.0 Design and Construction

During the planning and pre-design stages it had been decided to break the project into several components and contracts. This was due to the City's concern about having some basic temporary facilities in place until the new system was complete. The project was therefore broken into the following components:

Temporary Pumpout Facilities - By City Forces
Downstream Gravity Sewers - By City Forces
Temporary Lift Station & Storage Pond - By City Forces
Internal Gravity Sewers - By Private Sector
Lift Station and Storage Tank - By Private Sector

4.1 Construction by City Forces

In the Winter of 1989 the City proceeded with the design and construction of the temporary pumpout facilities, downstream gravity sewer improvements and the temporary lift station and storage pond. These improvements would provide a basic level of protection against basement flooding during the 1989 wet weather season until the new permanent facilities could be completed. The downstream gravity sewers were completed in the Spring of 1989 using conventional open cut excavation techniques.

The design of the temporary lift station and storage pond was based on the pump-in gravity-out flow equalization concept. Although not as reliable a concept as the one chosen for the permanent improvements it proved to be an economical interim solution. The lift station consisted of a 3.5 metre diameter corrugated structural plate caisson incorporating 2 pumps discharging into a temporary surface storage pond. The discharge structure from the storage pond consisted of a manually operated sluice gate to control flow into the downstream receiving sewer.

In June 1989 a storm event necessitated the use of those facilities. The use of temporary pumpout facilities in various manholes to pump diluted sewage to the storm drainage system proved to be controversial but succeeded in keeping basements dry during this storm.

4.2 Internal Gravity Sewers

As a result of the recommendations of the Pre-design Report it was decided that the Internal Gravity Sewers should be tendered to allow Tenderers to Bid on various methods of construction involving either conventional open cut techniques or trenchless techniques such as micro-tunnelling and auguring. Detailed plans and specifications were therefore developed toward two different pipe alignments which would allow the maximum flexibility for tenderers to bid on the construction method of their choice.

A major concern during the design of the gravity sewers was the disturbance of normal traffic and public transit routes during construction as well as the general environmental impact due to noise and other factors of the construction on the local residents. In order to implement the construction in a timely, controlled and efficient manner the internal gravity sewer improvements were broken into a series of 3 separate contracts. Tendering of the contracts was scheduled to facilitate staged construction thereby eliminating major disruption to traffic flow.

Contract No. 2 was tendered in July and awarded to the low tenderer, FMX Construction of Fort McMurray, Alberta on the basis of conventional open cut construction techniques. From the outset of this contract the contractor encountered problems with shifting excavation walls despite using trenching cages. The problem was complicated by the fact that the original trench backfill material had not been fully compacted and thus combined with the high water table created very unstable excavation conditions Pipe installation was eventually completed in November after a 3 month construction period.

The experience of Contract No. 2 further emphasized the importance of the serious consideration of trenchless pipe installation techniques on the other contracts. Also because of the sequencing of the contracts, conventional techniques would necessitate a 4 month delay in completing the construction due to backfilling and resurfacing limitations during the winter weather. The low tenderer for both Contracts Nos. 1 & 3 was IGL Construction Ltd. based on using the PIM (Pipe Insertion Machine) method for the majority of the pipe replacement. The PIM method involves a hydraulic bursting unit which shatters the existing pipe into the surrounding soil while enlarging the annular space. A new larger diameter high density polyethylene carrier pipe is pushed into place behind the bursting unit.

This technique had only been used for pipe replacement in Canada once before on a cast iron watermain replacement project in Regina. To further complicate analysis of the use of this method some portions of the two Contracts involved upsizing from a 250 mm diameter to a 450 mm diameter something that had not been previously accomplished using this technique. Based on research of its use in North America and the United Kingdom and a guarantee provided by the Contractor the method was approved and both Contracts eventually awarded to IGL

The use of the PIM technology allowed the contractor to work on both contracts over the winter months. It also allowed traffic to flow through the construction sites during the construction period, something which could not be achieved using conventional techniques. Other than opening the surface for pipe insertion and retrieval, and reconnection of service connections, the surface disruptions and repaving requirements were kept to a minimum. The success of the PIM method proved to be very dependent on the experience and skill of the operator, the type of pipe material being replaced, the travel speed of the insertion machine, and soil conditions. In two instances grade problems were encountered due to a combination of the above factors. In both cases the only suitable means of repair was open excavation and regrading of the pipe in the problem area. In all, approximately 1,175 metres of pipe were replaced using this technique with individual pipe runs varying from 58 to 123 meters in length. Open cut portions of these Contracts were completed in spring, 1990, thereby completing the construction of the internal gravity sewer rehabilitation.

4.3 Lift Station and Storage Tank

The site selected for the construction of the lift station and storage tank was an area approximately 36 meters in width and 240 meters in length. The site was bounded on one side by a 900 mm AWWA C301 concrete cylinder water feeder main and on the other side by a pole mounted overhead telephone line. With the basic configuration of the storage tank resolved, the focus of the design team addressed the geometric shape of each of the paralleling tubes. Various shapes were considered including circular and elliptical geometrics however in the end the recommended alternative was based on rectangular shaped tubes to take advantage of the reduced combination of structure width and height as well as simplicity of forming and construction.

Project scheduling dictated that the structure had to be constructed during the period from December to May. For this reason the design was focused toward simplifying winter construction techniques to the extent possible. It allowed for the possibility of using either cast-in-place or precast construction. The depth to the top of the structure was dictated by the depth of the incoming gravity sewer at 8 metres below ground surface. The structure was designed to dimensions of 4.5 meters high by 21.0 meters wide by 183 meters long.

The lift station wetwell depth was dictated by the depth of the storage tank. The design incorporated a full depth wetwell including three submersible pumps and a half depth drywell housing all discharge piping valving and instrumentation components for ease of access and maintenance. Overlying the wetwell and a portion of the drywell was a head tank incorporating influent piping from the pump discharge header, an effluent pipe complete with flow control valve and flow meter, and an overflow weir.

Under dry weather conditions the normal operating range of the wetwell would be below the bottom of the storage tank. Under wet weather conditions as the inflow exceeded the pumping capacity the level would rise filling the storage tank two cells a time The staged filling of the cells was controlled by incorporating overflow weirs at different elevations into each pair of cells and using flap gate check valves at the invert of each pair of cells to allow return flow to the lift station under drawdown condition.

the combined structure was tendered during the month of October. The Contract was awarded to the low tenderer PCL-Maxam construction Ltd. on the basis of cast-in-place construction techniques construction proceeded in December, 1989 with the first step being removal of the majority of the temporary storage pond to facilitate construction of the new facilities. A small section of the storage pond was left in place and the temporary lift station left operational to facilitate their use in the event of an overflow during construction period

PCL-Maxam developed an extremely efficient sequential construction technique which involved staged excavation, base preparation, base slab construction, roof and wall forming and pouring and staged backfilling for each 12 meter segment of the storage tank. Concurrent with the construction of the storage tank, progress continued on the lift station substructure and superstructure By early June the facilities were complete to the extent that testing, startups, and commissioning could proceed. Siteworks and landscaping were completed in late June and early July.

4.4 Hydraulic Testing

Once startups were complete, a hydraulic testing program was implemented to test the response of the lift station under various flow conditions in the 97th Street Trunk Sewer and vice versa. The results of this testing program were also used to calibrate the hydraulic model of the 97th Street Trunk Sewer, which was done as a separate project, and fine tune the settings for the control systems. The results of this hydraulic testing program proved the predicted conditions to be slightly conservative and thus only minor adjustments were required to the control system

5.0 CLOSURE

5.1 Community Involvement

Prior to implementing the project, basement flooding had been reported from well over 100 residences on several occasions. There was therefore a deep concern amongst the residents relative to the effectiveness of the existing sanitary sewer system and the City's plan for remedial action. A Flood Liaison Committee under the Lago Lindo Community League was established in 1988 to maintain ongoing liaison between the residents and the City and its Consultants This Committee met with the City representatives on numerous occasions during the investigation and design stages of the project to provide historical information and valuable inputs and suggestions to the design staff. It also took an active role in securing homes for field verification of the magnitude of infiltration under various ground moisture conditions. During the construction stage, the Committee was again instrumental in relating construction status and progress information to residents. At one point during construction when delays were encountered on completing construction on one street, a meeting involving all parties and residents living on both sides of the street was arranged to develop an action plan to minimize the inconvenience to the residents directly affected.

This ongoing means of community involvement from the very first day of the project to the end was found to be extremely useful. When the residents know what is going on and why certain decisions were made, they will tend to be more sympathetic and cooperative. This helped to reduce tension and conflicts between residents and the City Departments as well as contractors and avoid unnecessary delay to the completion of the project.

5.2 Validation of Design Parameters

Since the upgraded system was designed based on a 1 in 50 year designed storm event, it is important that the design parameters such as hydrographs and volume computations be verified. With this in mind, a number of Q-loggers were installed at strategic locations throughout the subdivision as well as along the downstream trunk to collect flow data for validation of the design parameters and assumptions. Valuable data were collected during the rainstorm events on June 27, 1988 and July 2, 1990, and in both occasions, the actual flow data measured were found to be very close to those simulated by the computer.

It is the City's intention to keep these flow monitors in place for a few more years to further validate the design parameters used in this project. This will also help to establish design criteria for I/I related calculations for other parts of the City which have similar characteristics.

5.3 Commissioning of the Upgraded System

All the construction works planned for the upgrading of the sanitary sewer system within Lago Lindo subdivision were completed on schedule before June 30, 1990. The final expenditure was calculated to be approximately $17 million which was 99% of the budgeted figure.

The upgrading system was officially commissioned on July 8, 1990. Members of the Lago Lindo Flood Liaison Committee and the press were all invited to attend the opening ceremony during which Major Jan Reimer lowered the last pump into position and activated the new pumping station.

EMERGING TRENDS IN WATER QUALITY STANDARDS

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IMPACT ON WATER TREATMENT PROCESSES

By

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Emerging Trends in Water Quality Standards
Impact on Water Treatment Processes

The past 15 years have seen a dramatic increase in public awareness of environmental issues in general and of water quality in particular. Virtually instant dissemination of information through the communications media, particularly television, has been largely responsible for this phenomenon. At the same time, bodies such as the American Water Works Association, the International Water Supply Association, the United States Environmental Protection Agency and various Ministries of the Environment have shown a greater willingness to cooperate in water quality related research and in the exchange of information.

The quality of the source water we tap is infinitely variable. It suffers both from the vagaries of nature and from man's interferences. It should not be surprising, therefore, that water treatment practices are still constantly evolving and are the subject of much attention.

Additional water quality concerns are constantly emerging. *Giardia lamblia* has received much attention. Recently, the AWWA publication *Opflow* identified Cryptosporidium as the water industries' "new superbug."[1]

Detection limits for micropollutants are steadily dropping. Advances in analytical equipment and continual reduction of detection limits have made it possible to set standards for substances which at one time were not known to exist in water supplies. Progress in epidemiology, toxicology and risk assessment, coupled with a growing concern over health risks, have resulted in water quality standards becoming more stringent with passing time. This trend is likely to continue, posing major challenges to water treatment facility designers and operators.

Water quality standards can be traced back far into antiquity. Vitruvias, a Roman architect and engineer related health and drinking water quality in the first century B.C..[2] Actual quantitative standards of water quality, except for infrequent references to aesthetics were, however, notably absent up to and including the nineteenth century. It was not until 1852 that a law was passed in London that all water thenceforth should be filtered. In 1914 the United States Public Health Service published the first regulatory water quality standards, and then initially only for application to interstate railroad service.[3] Canada's first comprehensive water quality guidelines, the Canadian Drinking Water Standards and Objectives was published in 1968.[4] The current Guidelines for Canadian Drinking Water Quality were established by National Health and Welfare in 1987 and issued in 1988.[5] It is considered that these guidelines are the most complete in use, although not necessarily the most stringent in all cases. Provincial regulatory authorities have, in general, adopted the guidelines as mandatory requirements. Some provinces have implemented mandatory filtration for surface water.

The purpose of this paper is to identify some of the more pressing water quality issues that are currently in the limelight and which may effect the design and operation of water treatment facilities now and in the near future. It is anticipated that pressure from the general public and special interest groups, the adoption of newer and more strict standards in other areas of the world and the influence of our own water supply industry, will cause us to have to search for improved treatment processes.

A case in point is the very real concern over lead levels in drinking water at the point of consumption. The current Maximum Acceptable Concentration (MAC) for lead is 50 g/L. This standard is common to Canada, the United States and the European Community.

Public concern over lead levels in drinking water in the United States caused the USEPA to propose a new Lead and Copper Rule in August 1988. The MCL proposed by the USEPA for lead entering the distribution system is 5 g/L.[6] Lead in drinking water at the tap will be regulated at 10 g/L in first draw samples (average) or 20 g/L in less than 5% of draw samples. Proposed no action levels (NAL's) are:

- pH> 8.0 in 95% of all samples
- average lead level <10 g/L
- lead level <20 g/L in 95% of all samples

If the NAL's are exceeded a public education program must be initiated and also a program to reduce levels put in place.

A stronger corrosion control regulation based on regulating pH and alkalinity leaving the treatment plant is anticipated. The regulation will be aimed at copper and lead as target corrodible materials.

Although the statutory deadline for promulgation of the Lead and Copper Rule was June, 1989, the setting of a MAC at the consumers tap has created so much controversy that it has been set back.[7] The USEPA is now committed to having a final lead rule by December, 1990. The Agency is reported to now be leaning towards setting a treatment technique rather than a maximum contaminant level.[8]

The proposed Lead and Copper rule will greatly affect the way water utilities do business, particularly in the distribution system. Additional unit processes may be required in the plants for water conditioning such as pH adjustment and alkalinity control. Polyphosphate additives may also be required in some instances.

Although this proposed rule is only applicable in the United States, it is certain to influence regulatory requirement in Canada. A new lead guideline of 10 µ/L, based on flushed samples at the tap, has been recommended by the Canadian Federal/Provincial Subcommittee on Drinking Water and is included in the Fourth Edition of the Guidelines for Canadian Drinking Water Quality.[9]

The USEPA has also presented a proposal that lead be given a B2 classification as a probable human carcinogen. This would establish the maximum contaminant level goad (MCLG) for lead at zero.[7]

As with lead and copper, a major source of cadmium in drinking water comes from corrosion of home plumbing systems. This element could be subjected to regulation in the future as a corrosion by-product.[7]

If Canada adopts the USEPA EPA's definition of primary disinfection as the inactivation of 3-logs of Giardia lamblia and 4-logs of enteric viruses it will become necessary in some plants to add additional contact tanks for disinfection or to move the application of lime to the downstream end of the plant. This is due to reduced disinfection efficiency at elevated pH levels. It should be recognized, however, that this approach could significantly increase turbidity in the finished water.

Elevated levels of aluminum in drinking water and the reported link between this and Alzheimer's Disease reported in the medical journal The Lancet, has caught the attention of the public.[10] Although the American Food and Drug Administration and the Alzheimer's Association have downplayed the reported link, pressure continues to climb from consumers and recognized authorities to reduce aluminum residuals in treated water.[11]

Regulatory standards for aluminum based on risk to health do not exist. The European Community has, however, set 0.2 mg/L as a maximum contaminant level based on aesthetics and the USEPA is proposing a Secondary Maximum Contaminant Level of 50 g/L based on post-precipitation in the distribution system.[12]

It is anticipated that control of aluminum residual will continue to be an issue with many water utilities. In some cases this may mean converting to a primary coagulant other than aluminum sulphate such as ferric chloride. In other cases it may require optimization of the coagulation process. This could include an acid feed system for pH adjustment prior to the addition of alum.

The balance of the inorganic contaminants that USEPA proposes to regulate are much in keeping with the existing Canadian Guidelines, although asbestos will be assigned an RMCL of 7.1 million fibres per litre. This will again influence treatment to reduce the aggressiveness of some waters.

Trihalomethane production by the chlorination of water containing certain organic constituents has received much attention. Chloroform, one of the more common THM's is a known carcinogen and the balance of the compounds are suspected to be cancer-causing.

At present, the Canadian Guidelines have set a maximum contaminant level for total trihalomethanes at 350 g/L, however, this level is currently under review. The USEPA limit is 100 g/L. The European

Community has set the chloroform MCL at 30 g/L. Michael B. Cook, director of the USEPA Office of Drinking Water, has stated that the Agency is considering a new MCL for trihalomethane in the range of 25-50 g/L instead of the current 100 g/L.[8]

The new USEPA proposed regulations assume that any amount of a carcinogen in the water is unacceptable. It is from this philosophy that a value of 0 has been established for Recommended Maximum Contaminant Levels (RMCL's) for certain Volatile Organic Compounds. On this same basis, one might reasonably expect the new RMCL's for each trihalomethane (or at least for chloroform) to be set at 0. One can further argue that the actual Maximum Contaminant Levels for THM's might be in the order of 5 g/L since MCL's are required by law to be set as close as feasible to the RMCL's.[13] In fact, current discussion does not preclude this. Peter L. Cook, deputy director, USEPA Office of Drinking Water made the following statement: "If the substance under investigation is a known or probable carcinogen, the maximum contaminant level goal will be set at zero as a matter of Agency policy. The MCL is then set as close to the MCLG as feasible."[14]

The eventual adoption of such low contaminant levels will significantly affect the water treatment processes for many plants. The three approaches to controlling trihalomethanes are: remove the precursors; remove the formed trihalomethanes, or; employ oxidants or disinfectants other than free chlorine.

The use of air stripping columns for the removal of THM's and their precursors will probably accelerate as will the use of activated carbon adsorption techniques. However, probably the largest shift in treatment approaches will be away from the use of free chlorine and toward the use of alternative oxidants such as chlorine dioxide, monochloramine, and ozone. In view of the concerns about the possible toxicity of chlorite and chlorate, which are reaction products of chlorine dioxide, and the reduced disinfectant capacity of monochlorine, the use of ozone will emerge as the primary oxidant of choice.

In 1987 the USEPA promulgated maximum contaminant levels for eight Volatile Organic Chemicals:

Tetrachloroethylene
carbon tetrachloride
1,1,1-trichloroethane
vinyl chloride
1,2-dichloroethane
benzene
1,1-dichloroethylene
p-dichlorobenzene

At the same time EPA specified that the Best Available Technology for the removal of these VOC's is Packed Tower Aeration or Granular Activated Carbon for all but vinyl chloride. These treatment techniques will be employed in increasing numbers for contaminated ground water sources.

The USEPA has also announced proposals to regulate 38 substances including PCB's. Of these, 27 are being regulated in drinking water for the first time.[15]

Advanced oxidation processes such as the combination of ozone and hydrogen peroxide or ultra-violet light to promote oxidation and mineralization of trace organics is showing much promise.

Attention is now being focused on other disinfection byproducts such as haloacids, halonitriles chloropicrin, chloral hydrage, cyanogen chloride, MX and N-chloramines. Again, the culprit is free chlorine. Health effects are being determined for these compounds and MCL's are anticipated for proposed regulation in the United States by means of a complex Disinfection-Disinfection By-products Rule to be introduced in late 1991 or early 1992.[8] This will no doubt again stimulate the use of ozone as a primary oxidant.[16]

Ozone creates its own oxidation byproducts at the low levels used for water treatment such as low molecular weight aldehydes. These have not, however, been demonstrated to significantly affect human health.

The use of ozone can result in an increase in the amount of assimilable organic carbon (AOC) entering the distribution system. This AOC provides the substrate for biological activity thus requiring post chlorination. The use of chloramination for post disinfection will retard the formation of THM's in the system while providing disinfectant properties and will become increasingly popular.

A secondary benefit of ozone is that it frequently assists in the coagulation-flocculation process.

The use of the C.T concept is employed in the USEPA's Surface Water Treatment Rule. The C.T value is the numerical product of disinfectant concentration multiplied by time of contact. Specific C.T values have been established for various disinfectants based on inactivation of Giardia lamblia cysts and enteric viruses.

The contact time is not the theoretical detention time in the contact basin or reservoir but rather the actual flow-through time. It is quite probable that many existing facilities suffer from short circuiting and could not comply with C.T requirements without excessive disinfectant dosages. In a number of instances additional reservoirs or contact chambers will have to be constructed if this requirement is introduced here.

C.T values for ozone range from a maximum of 3 mg/L - min at 0.5°C down to 0.67 mg/L - min at 25 °C. This means that 99.9% destruction of Giardia lamblia cysts or 99.99% destruction of enteric viruses will occur at 0.5°C with an ozone dose of 0.5 mg/L and 6 minutes of contact time. To obtain the same disinfectant results with free chlorine at a pH of 7.0 and a concentration of 2 mg/L would require a contact time of over 2 hours.

Since prefiltration oxidation is credited to the C.T value, the use of ozone would generally offset the need to construct additional contact storage.

The reduction of turbidity to less than 0.1 turbidity units will be pursued even more actively in the future. Increased attention will be paid to optimizing the prefiltration processes in order to reduce the load on the filters. This will include pH adjustment, optimization of coagulant dosing by use of various polyelectrolytes, the use of dissolved air flotation in place of gravity sedimentation for many waters and the use of fully computerized control systems. The more-or-less abandoned practice of filtering to waste immediately after putting a filter on line will regain popularity. Particle counting and size distribution monitoring will become more prevalent.

Consumers are rightfully demanding increased taste and odour quality. This will further increase the application of ozone to eliminate chlorinous tastes and odours and to destroy taste and odour compounds such as geosium and 2-methylisoborneol. The use of activated carbon either in powdered or granular form will increase.

Radon gas in ground water supplies is becoming a greater concern and has caught the attention of the consumer. Air stripping and granular activated carbon systems will become more prevalent where radon is found to exist.

Increased importance will be placed on remote up-stream monitoring and on broad spectrum biological monitors to provide advance warning of water quality changes such as spills of toxic substances. Watershed management will become more critical.

The Canadian Federal Government is now putting the finishing touches on long-discussed legislation that will likely be known as the Drinking Water Safety Act. Once passed, the Canadian Drinking Water Guidelines will be renamed "objectives" and will be mandatory requirements in areas that fall under federal jurisdiction.

The Canadian Water and Wastewater Association published comments on the 1987 Guidelines which, among other things, recommended additional parameters for consideration in the future. In all probability, the regulatory authorities will continue to apply increasingly stringent standards to the production of drinking water. As with many environmental issues, some of the requirements will be technically justified, however, some decisions may be made in the public arena without reliable cost-benefit analyses. In any event, the 1990's promises to be unparalleled in study, design and construction activities in the continuing quest for safer drinking water.

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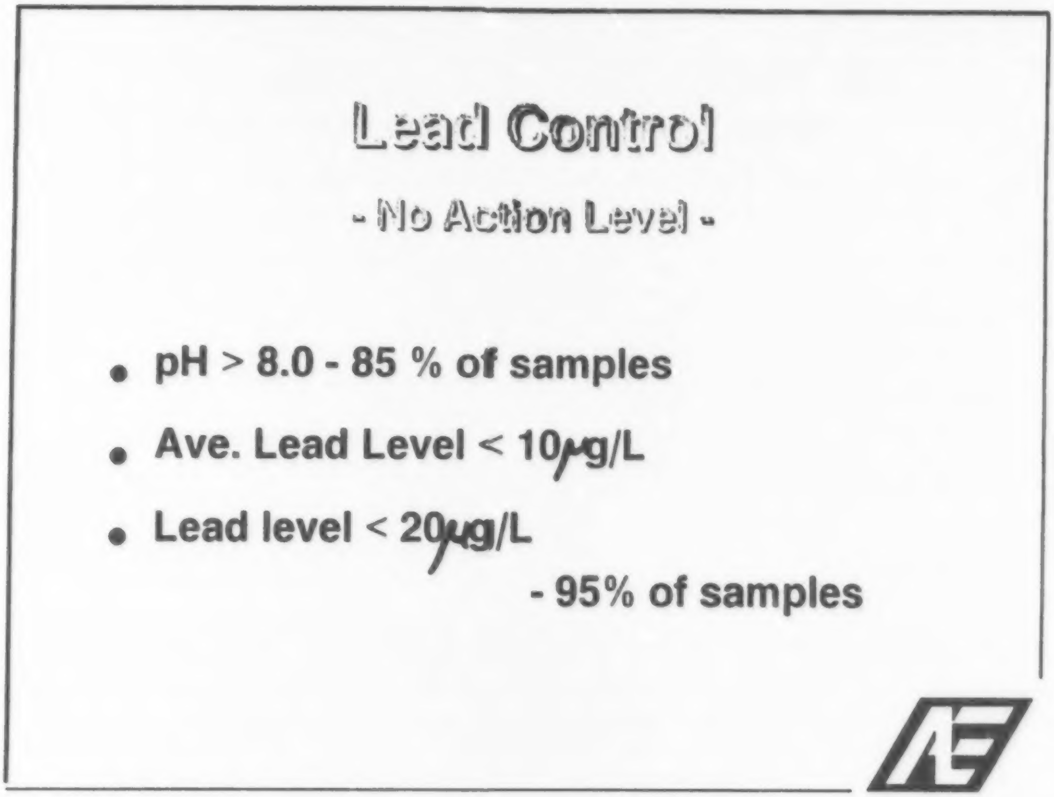
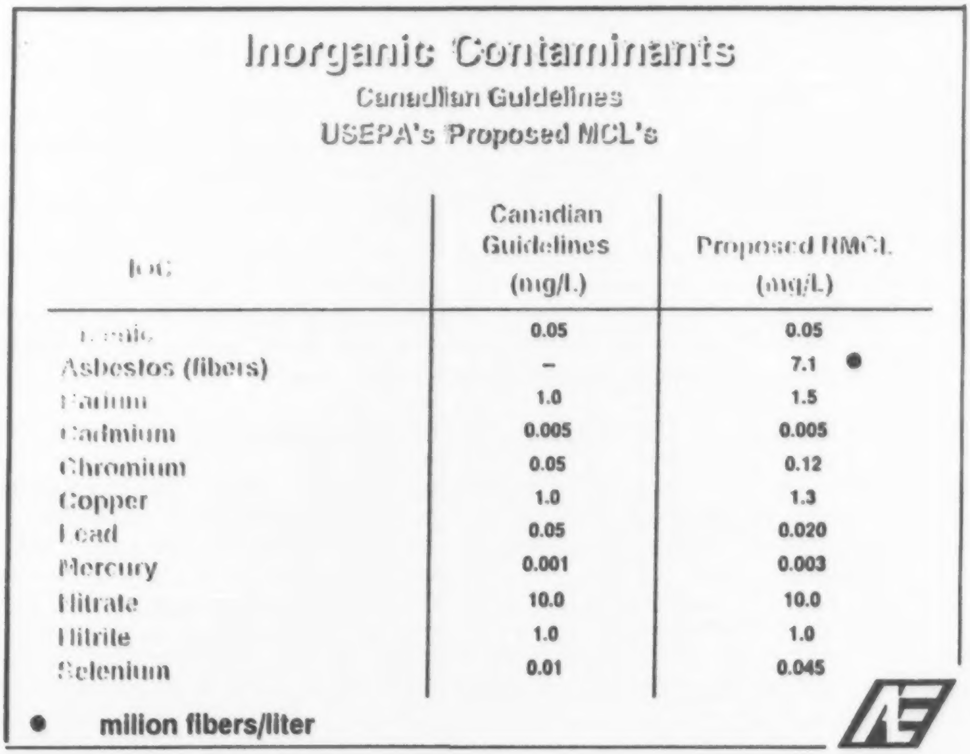


Lead Control

- No Action Level -

- pH > 8.0 - 85 % of samples
- Ave. Lead Level < 10µg/L
- Lead level < 20µg/L
  - 95% of samples



Inorganic Contaminants

Canadian Guidelines
USEPA's Proposed MCL's

IOC	Canadian Guidelines (mg/L)	Proposed RMCL (mg/L)
Arsenic	0.05	0.05
Asbestos (fibers)	–	7.1 ●
Barium	1.0	1.5
Cadmium	0.005	0.005
Chromium	0.05	0.12
Copper	1.0	1.3
Lead	0.05	0.020
Mercury	0.001	0.003
Nitrate	10.0	10.0
Nitrite	1.0	1.0
Selenium	0.01	0.045

● milion fibers/liter

Trihalomethanes

- Chloroform
- Bromodichloromethane
- Dibromochloromethane
- Bromoform

Volatile Organic Chemicals

Canadian Guidelines
USEPA's Proposed MCL's

VOC	Canadian Guidelines (mcgm/L)	Proposed MCL (mcgm/L)
Benzene	5	5
Vinyl Chloride	–	1
Carbon Tetrachloride	5	5
1,2 Dichloroethane	–	5
Trichloroethylene	–	5
1,1 Dichloroethylene	–	7
1,1,1-Trichloroethane	–	200
p Dichlorobenzene	–	750

Synthetic Organic Contaminants

Canadian Guidelines

USEPA's Proposed MCL's

SOC	Canadian (mg/L)	Proposed RMCL (mg/L)
Acrylamide	–	0
Alachlor	–	0
Aldicarb, aldicarb sulfoxide and aldicarb sulfone	0.009	0.009
Carbofuran	0.09	0.036
Chlordane	0.007	0
cis-1,2-Dichloroethylene	–	0.07
DBCP	–	0
1,2-Dichloropropane	–	0.006
o-Dichlorobenzene	–	0.62
2,4-D	0.1	0.07
EDB	–	0
Epichlorohydrin	–	0

(continued)

Synthetic Organic Contaminants

Canadian Guidelines

USEPA's Proposed MCL's

	Canadian (mg/L)	Proposed RMCL (mg/L)
[illegible]	0.0024	0.68
[illegible]	--	0
[illegible]	--	0
[illegible]	0.004	0.0002
[illegible]	0.9	0.34
[illegible]	--	0.06
[illegible]	--	0.22
[illegible]	--	0.14
[illegible]	0.024	2.0
[illegible]	--	0.052
[illegible]	--	0
[illegible]	--	0.07
[illegible]	0.3	0.44

CT Values for 99.9% Inactivation of G. lamblia Cysts

Disinfectant	pH	0.5 °C	10 °C	20 °C	25 °C
Free Chlorine @ 2 mg/l	6	171	91	46	30
	7	261	140	70	47
	8	377	201	101	67
	9	521	279	139	93
Ozone	6-9	3	1.67	1	0.67
Chlorine Dioxide	6-9	54	27	14	9.30
Chloramine (preformed)	6-9	3,300	1,850	1,100	750

OVERHEAD #6

PNEUMATIC

ADVANTAGES

- valve can fail to either direction upon power loss
- infinite modulation
- air is clean and relatively low cost
- usually less expensive
- simpliest form of actuator
- ease of maintanence
- generally trouble free
- ease of field repair availablity

DISADVANTAGES

- can be big and bulky
- air is compressible
- requires a compressed dry air supply
- upon loss of air, difficult to manually override

OVERHEAD #7

ELECTRIC

Advantages

- can be used where no air is available
- has torque sensing capabilities
- has intregal manual override

Disadvantages

- upon power loss, valve stays in last position
- complex piece of equip. Some cases not easily field repairable

OVERHEAD #8

HYDRAULIC

Advantages

- Valve fail to either direction upon power loss
- capable of very high torque
- liquid is not compressabale
- capable of very high torque

Disadvantages

- can be costly to maintain
- fluctuations in supply pressure affect torque
- can be messy if cylinder leaks
- difficult to manually override

AEROBIC TREATMENT OF MUNICIPAL WASTEWATER COMBINED WITH EFFLUENT FROM AN ANAEROBIC PRETREATMENT FACILITY: PILOT SCALE INVESTIGATION

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Introduction

The preferred wastewater treatment option for the city of Portage la Prairie, Manitoba was tested from August, 1989 to March, 1990 using pilot-scale reactors to simulate the process. In particular, it was necessary to test how the anaerobic and aerobic biological processes would respond to the unique wastewater characteristics of the City. The study determined important process design information.

The success of the pilot study would not have been achieved without team-work, which involved several different entities. The team consisted of the following key components: the University of Manitoba, Environmental Engineering Division staff, who maintained and operated the pilot plant; the City of Portage la Prairie staff, who responded to numerous emergencies; the ADI Limited staff, who ensured the consistent operation of the anaerobic reactor; the Technical Committee, who's key decisions at critical times helped keep the study on track. Poetker MacLaren staff provided technical coordination of all groups involved in the study.

Objectives

The preferred treatment option was comprised of a conventional activated sludge system to treat the domestic wastewater combined with the effluent from a low rate anaerobic system which will pretreat the Industrial Park wastewater. The recommended low rate anaerobic system was the Bulk Volume Fermenter (BVF^{TM}), marketed by ADI of New Brunswick, provided that it was capable of achieving the following:

1) Successful pilot scale testing, carried out in Portage la Prairie to verify the performance of the BVF, to confirm that the proposed system components will address solids carryover; and

2) Successful concurrent pilot scale testing of an activated sludge (AS) system treating a combination of effluent from the anaerobic treatment process and the municipal wastewater.

To satisfy these process performance conditions, an 8 month pilot scale treatability study was conducted. The detailed objectives of the study were as follows:

1) To determine the organic removal efficiency for the BVF under various temperatures and hydraulic loading rates.

2) To determine chemical dosage requirements for pH adjustment in the BVF;

3) To determine the optimum food to microorganism (F:M) ratio or solids retention time (SRT) and hydraulic retention time (HRT) of the AS process;

4] To determine the optimum sizing of the BVF and the AS reactors.

The BVF was operated at HRT's of 10 and 6 days, which approximate the projected 5-year average dry weather flow (ADWF) and the 5-year 7-day sustained peak flow, respectively, using a 50,000 m^3 reactor as recommended by ADI. The 5-year 7-day sustained peak flow was estimated to be 1.8 times the 5-year ADWF. This was used because the BVF should not be significantly affected by short duration peak flows. The AS reactors were operated at the 5-year peak dry weather flow (PDWF) for both the industrial and municipal flow components. This resulted in a municipal wastewater to BVF effluent ratio of 1.9 to 1, which represents the highest proportional BVF effluent load expected during the design life. The optimum SRT and HRT for the AS system was determined at this flow ratio.

Equipment & Monitoring Program

The anaerobic BVF supplied by ADI was initially operated with a single overflow baffle. The poor performance of the reactor led to the modification of the BVF, during week 17 of the pilot study when two underflow baffles were installed.

The AS reactors and secondary clarifiers were also modified during the course of the pilot study. The original equipment consisted of separate aeration basins and secondary clarifiers, such that the effluent from the aeration basin flowed by gravity into the upflow clarifier. The settled sludge was recycled from the bottom of the clarifier back to the aeration basin, however, the recycle lines frequently clogged and caused severe operational problems. The initial aeration basins and clarifiers were consequently replaced with unified reactor-clarifiers, which eliminated these problems.

The process flow diagram of the pilot plant is shown in **Figure 1.** A 24-h composite sample was collected at the Industrial Park lift station and transported to the pilot plant. The wastewater was then manually transferred through a fine screen into the storage tank. The compartmentalized BVF was fed several times per hour, 24 hours per day, using a timer and pump system. An internal recycle system was used. During the final eleven weeks of operation, the recycle line was connected to the influent line to increase sludge/biomass contact and was operated on an intermittent basis. The effluent was pumped through a preaeration tank and clarifier before being discharged into the activated sludge feed tank. The municipal wastewater flowed continuously into a storage tank followed by a primary clarifier. The municipal flow was then combined with the industrial flow in the activated sludge feed tank. The feed was pumped to the unified activated sludge reactors using four separate pumps. The unified reactors consisted of an aeration basin and an upflow secondary clarifier joined by an underflow baffle similar to that presented in Metcalf and Eddy, 1979.

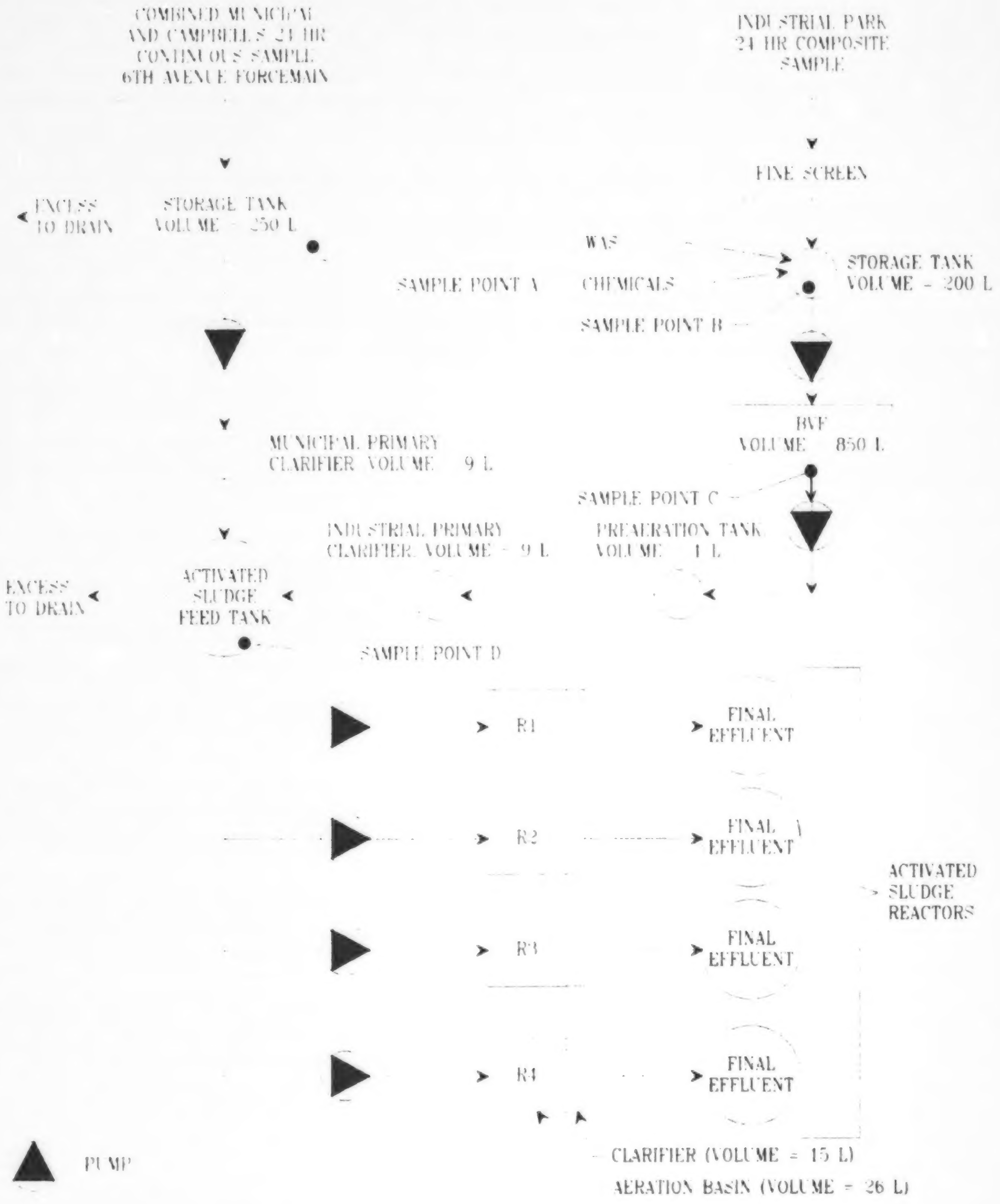
COMBINED MUNICIPAL
AND CAMPBELLS 24 HR
CONTINUOUS SAMPLE
6TH AVENUE FORCEMAIN
INDUSTRIAL PARK
24 HR COMPOSITE
SAMPLE
FINE SCREEN
EXCESS
TO DRAIN
STORAGE TANK
VOLUME = 250 L
WAS
CHEMICALS
STORAGE TANK
VOLUME = 200 L
SAMPLE POINT A
SAMPLE POINT B
BVF
VOLUME = 850 L
MUNICIPAL PRIMARY
CLARIFIER VOLUME = 9 L
SAMPLE POINT C
INDUSTRIAL PRIMARY
CLARIFIER VOLUME = 9 L
PREAERATION TANK
VOLUME = 1 L
EXCESS
TO DRAIN
ACTIVATED
SLUDGE
FEED TANK
SAMPLE POINT D
R1
FINAL
EFFLUENT
R2
FINAL
EFFLUENT
R3
FINAL
EFFLUENT
R4
FINAL
EFFLUENT
ACTIVATED
SLUDGE
REACTORS
PUMP
CLARIFIER (VOLUME = 15 L)
AERATION BASIN (VOLUME = 26 L)

FIGURE 1. Process flow diagram of the pilot plant. Sample points correspond to data given in Table 2.

The temperature of the BVF was controlled using insulation and heat tracing tape controlled by thermal switches. The temperature of the AS reactors were controlled using a water bath.

The pilot plant was located on the second level of a wastewater lift station. Analytical work was conducted on the ground level of the lift station and at the UM's Environmental Engineering laboratory, in Winnipeg.

The general sampling and analysis program for the BVF and AS reactors are outlined in **Tables 1 and 2** respectively.

Results and Discussion

The general chemical properties of the various wastewater components during the study are presented in **Table 3.** The sampling points have been indicated in **Figure 1.** The BVF results will be discussed first, followed by the AS results of the pilot study.

BVF Performance

The removal of organics (COD or BOD) and suspended solids (SS) were evaluated at the temperature and hydraulic loading (organic space loading) rates expected during the full-scale operation of the BVF.

The BVF achieved 80% COD removals during the 26th week of operation (**Figure 2**). Removal efficiencies before this week were inconsistent and were generally lower than 60%. The twenty-six weeks were needed to build-up an active acclimated biomass capable of utilizing the incoming organic load. It was suspected that the high effluent SS concentrations, before the BVF modifications of week 17, allowed the biomass to escape with the effluent (**Figure 3**). After the modifications (week 17), the biomass was retained in the BVF and allowed to build-up, which resulted in an effluent COD decrease from weeks 21 to 30 (**Figure 4**). The low effluent COD values from week 26 on, allowed $\geq$ 75% organic removal efficiency at influent COD concentrations of 2,500 to 11,000 mgL^{-1}.

Nutrient supplements to the BVF were stopped after week 25 and the temperature was maintained at 20^0C for week 28 until the end of the study (**Figure 2**). Waste activated sludge (WAS) additions of 5% and 10% of the influent COD concentration were found to have no affect on the effluent SS (Figure 3) or the effluent COD (**Figure 2**).

The data from weeks 26 to 31 and from 32 to 34, inclusive, were used as the basis for the performance of the BVF at HRT's of 10 days and 6 days, respectively. This performance data is summarized in **Table 4.**

The 10 and 6 day HRT's correspond to the 5 and 20 year design loads. The organic loading rate (OLR) increased 4-fold due to the increased flow and influent COD concentration (0.32 to 1.29; **Table 4**) during the final three weeks of the study. The increased removal efficiencies at the shorter HRT of 6 days was due to the very high influent organic and SS concentrations during the last three weeks. The effluent concentrations during the same study periods are given in **Table 5.**

TABLE 1. The General BVF Sampling and Analysis Program conducted during the Pilot Study

PARAMETER	INFLUENT	EFFLUENT
pH; T; Q; Q_G	daily	daily
COD; BOD_5; SS; VSS	weekly composite	weekly composite
Volatile acid alkalinity; Total alkalinity	2 - 3 per week	2 - 3 per week
TKN; NH_3-N; TP; SO_4; soluble sulfide; FOG	monthly	monthly
Biogas composition (CO_2; H_2S)	weekly	weekly

TABLE 2. The General Activated Sludge Sampling and Analysis Program conducted during the Pilot Study

PARAMETER	INFLUENT	REACTOR	EFFLUENT
pH; T; Q; DCOD; SOC; SS	daily		daily
COD; TOC	daily		
MLSS; SSV; SVI; DO		daily	
$DBOD_5$	5 per week		5 per week
TKN; NH_3-N; NO_X-N; TP; SO_4; soluble sulfide	weekly		weekly
BOD_5	5 per week		
OUR; ZSV		weekly	
VSS; MLVSS	weekly	weekly	weekly

TABLE 3. Wastewater Chemical Characteristics Measured at Various Sampling Points (Sampling Points are Show Figure 1)

Sampling Point	BVF Influent (A) (mgL^{-1})	Municipal Sewage** (B) (mgL^{-1})	BVF Effluent (C) (mgL^{-1})	Activate Sludge Influent** (D) (mgL^{-1})
COD	4970 ± 2640	472 ± 137	940 ± 280	406 ± 137
DBOD$_5$	2390 ± 760		590 ± 145	215 ± 67
SOC	1318*	159 ± 148	378*	169 ± 68
Ammonia	111.2	11.9 ± 2.5	202.1	57.0 ± 16.7
TKN	204.7	38.1 ± 15.5	205.7	69.8 ± 16.1
Total Phosphorus	43.3	5.75 ± 2.8	21.8	11.0 ± 4.9
Sulfate (as S)	40.0	69.0 ± 32.5	26.7	49.6 ± 21.8

* TOC instead of SOC

** Average values of all samples from Jan.06 to Mar.30.

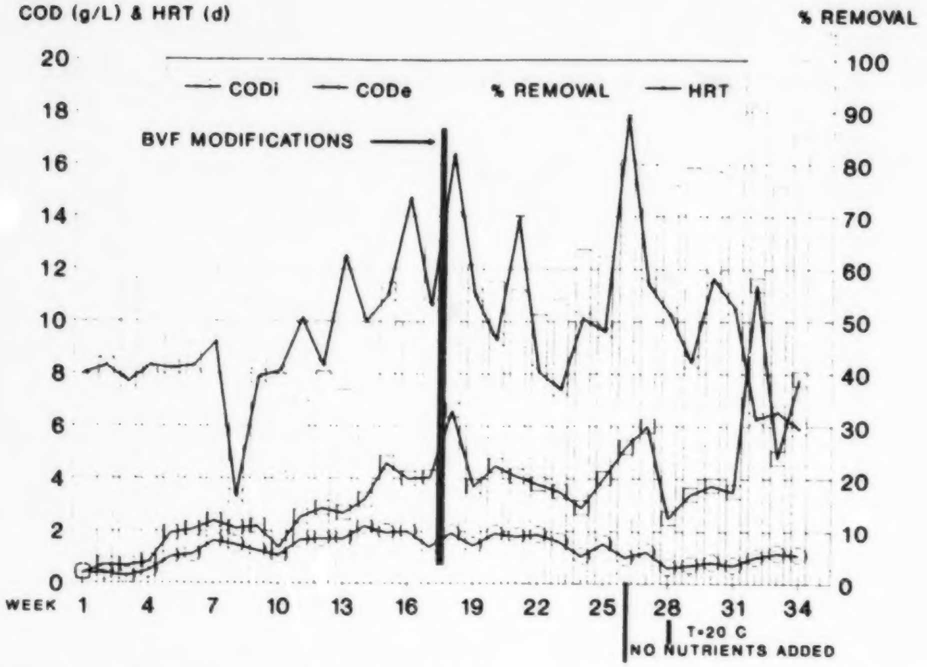

FIGURE 2. Plot of the influent COD (CODi), the effluent (CODe), the COD removal efficiencies, and the weekly average hydraulic retention times (HRT) in the BVF during the pilot study. No micro or macro were added to the BVF after week 25. The BVF operated at 20°C from weeks 28 to 34, inclusive.

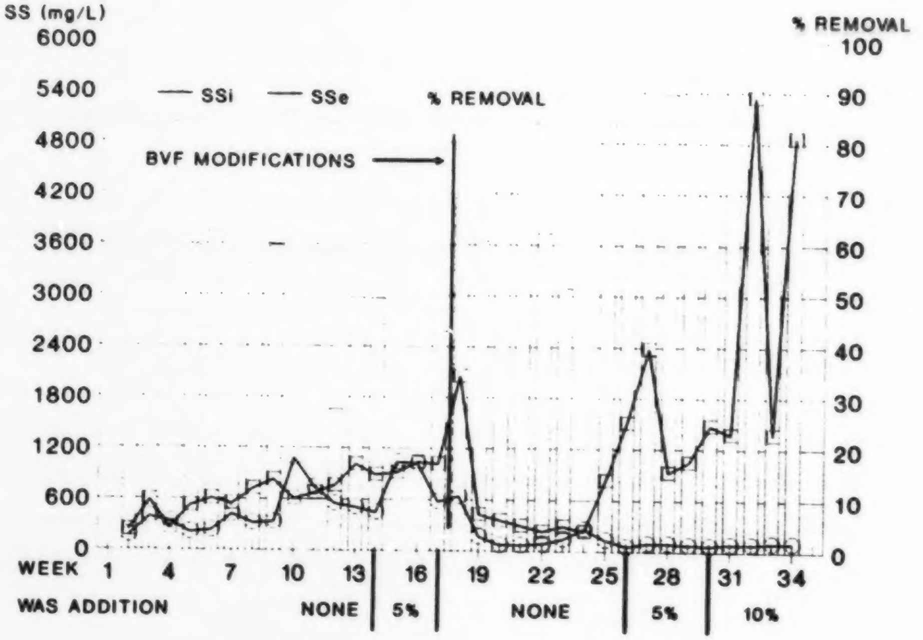

FIGURE 3. Plot of the influent SS (SSi), the effluent SS (SSe) and the SS removal efficiencies in the BVF during the pilot study. Waste activated sludge (WAS) loads indicated as percentage of influent COD concentration.

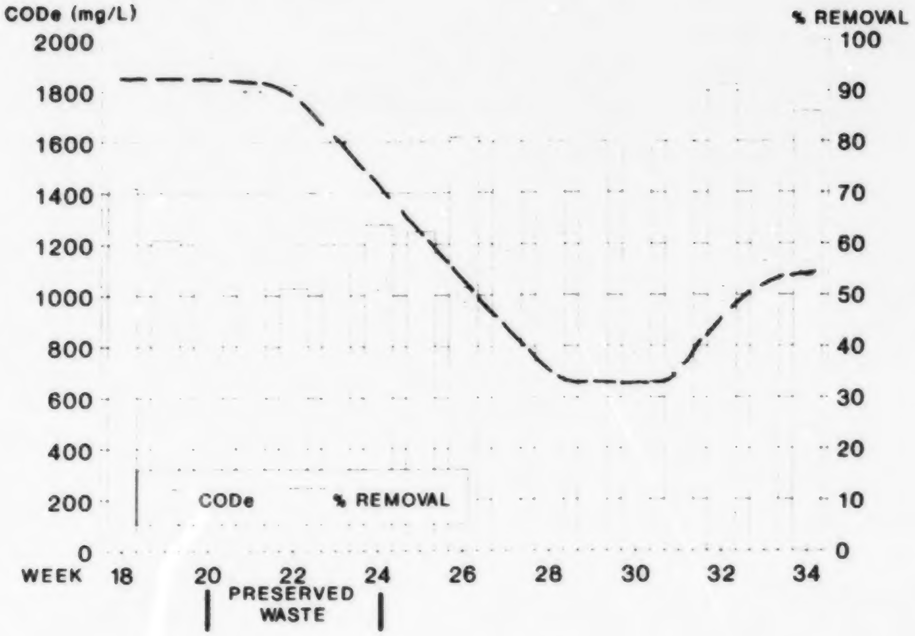

FIGURE 4. Plot of the effluent COD (CODe) and COD removal efficiences of the BVF during weeks 17 to 34 of the pilot stu
Period of preserved waste use due to McCain's shutdown.

TABLE 4. Performance of the BVF at the Two Hydraulic Retention Times (HRT's) Investigated during the Pilot Study

WEEKS	HRT	OLR	REMOVAL EFFICIENCIES (%)		
(inclusive)	(d)	(gCOD $L^{-1}d^{-1}$)	COD	BOD	SS
26-31	10	0.32	78.5	81.8	93.9
32-34	6	1.29	84.3	81.8	96.3

TABLE 5. Effluent quality of the BVF During the 10 and 6 Day HRT periods of the Pilot Study

HRT (d)	COD$_e$ (mgL^{-1})	BOD$_e$ (mgL^{-1})	SS$_e$ (mgL^{-1})
10	833 ± 225	*568 ± 174	81 ± 13
6	1087 ± 81	**501 ± 187	94 ± 9

* Carbonaceous and nitrogenous BOD$_5$.

** Carbonaceous BOD$_5$ only.

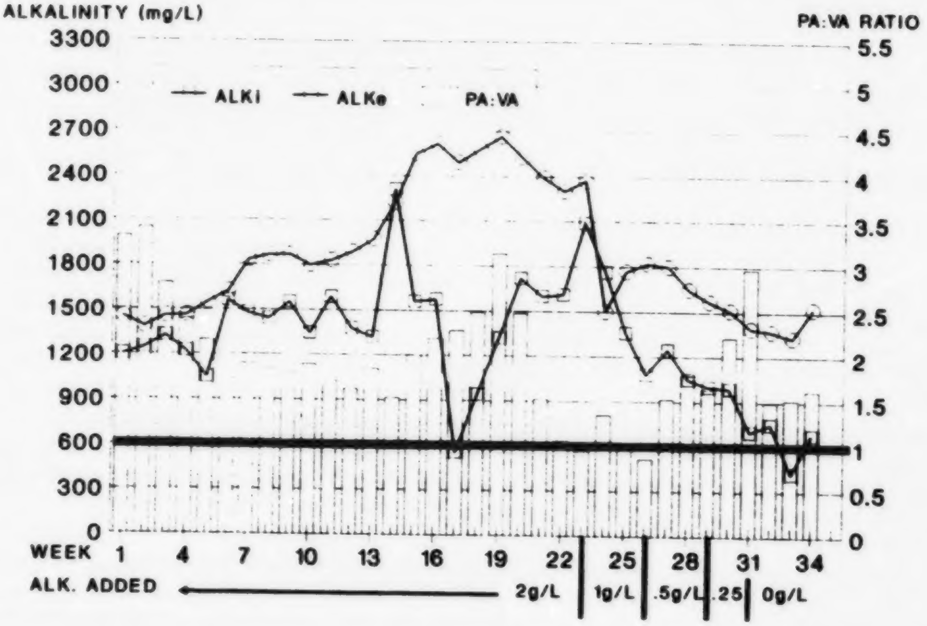

FIGURE 5. Plot of the influent alkalinity (ALKi), the effluent alkalinity (ALKe), and partial alkalinity (PA) to volatile acid alkalinity (VA) ratio in the BVF during the pilot study. PA:VA ratio above 1 indicates a stable process. Alkalinity supplements (as NaHCO$_3$) indicated along horizontal axis.

Chemical Supplements

The sodium bicarbonate (N_aHCO_3) alkalinity supplements to the BVF were lowered during the last twelve weeks of the study (**Figure 5**). The influent alkalinity concentration dropped during the lowering of the supplement, but stabilized at 613 ± 260 mgL^{-1} as $CaCO_3$ during the final four weeks of operation, when no alkalinity supplements were added. The effluent alkalinity concentration also dropped during this time period, but appears to have stabilized at 1393 ± 119 mgL^{-1} as $CaCO_3$ during the final four weeks. The stability of the alkalinity-buffering system of an anaerobic reactor is usually measured by relating the volatile acid concentration to the alkalinity in the reactor. ADI achieves this by relating the partial alkalinity (PA) to the volatile acid alkalinity (VA). The VA is the component of the total alkalinity attributed to neutralizing the volatile acids, while the PA is the difference between the total alkalinity and the VA. Using this nomenclature, there is sufficient buffering capacity in the BVF when the PA:VA ratio is above 1. Simply stated, this means that the alkalinity to volatile acid ratio is in a stable equilibrium. The PA:VA ratio was above 1.5 during the final eight weeks of operation which indicates a stable reactor (**Figure 5**).

These results suggest that alkalinity supplements will not be needed after the start-up period, however, it is difficult to state conclusively, due to the short operation time without alkalinity supplements.

Influent nutrient supplements were not added to the BVF during the last nine weeks of the study. No relationship between nutrient addition and process performance was observed. This indicated the addition of nutrients had no distinguishable benefits and were not required.

Waste Activated Sludge Addition

The Waste Activated Sludge (WAS) additions were controlled based on influent COD concentrations. The COD of the WAS added to the influent was 5% of the total influent COD concentration during weeks 26 to 29 and 10% during weeks 30 to 34 (**Figure 3**) These loads caused the influent SS to increase, but had no effect on the effluent SS concentration, or the effluent COD concentration. The WAS load at 10% of the influent COD allows for all WAS produced during the 20 year design period to be treated in the BVF.

The preaeration tank operated at an HRT of 10 to 20 minutes and used an air:liquid ratic of 100 LL^{-1}. This short aeration period allowed for 26% removal of the dissolved sulfides. The sulphide concentration of the combined municipal:industrial activated sludge feed was 3.6 ± 1.1 mgL^{-1} during the last 5 weeks of the pilot study.

The preaeration unit operation introduces oxygen into the BVF effluent and also strips hydrogen sulfide gas. The latter is important for control of odour, corrosion and thiothrix species of filamentous bacteria. The BVF effluent dissolved sulfide concentration was high and variable (20 ± 16.7 mgL^{-1}), therefore the use of a preaeration facility was considered necessary.

Activated Sludge Performance

General

In order to satisfy the objectives of the pilot study, the activated sludge testing was separated into two phases. The first phase maintained a constant HRT in all four reactors, while the solid retention time (SRT) were varied. The second phase maintained the optimum SRT of phase one in all four reactors, while HRT varied. The operating conditions of each reactor for the two phases and the durations of each phase are presented in **Table 6**. The F:M values were calculated during the pseudo steady-state conditions.

The removal of organics was not greatly affected by the different SRT's. The BOD_5 removal efficiencies ranged from 93 to 96 %, while the effluent BOD_5 concentrations ranged from 9.5 to 12 mgl^{-1}, despite influent BOD_5 variations from 53 to 383 mgl^{-1}. This indicates that the organics were removed easily at all SRT's investigated. The soluble BOD_5 removals observed in R1 are presented in **Figure 6**.

The treated wastewater will be discharged into the Assiniboine River. Due to the limited ammonia receiving capacity of the River, a long SRT will be needed to ensure nitrification of the wastewater. For this reason, along with reduced WAS production, the 12 d SRT was selected as the optimum SRT.

Optimum HRT

The second experimental phase used the optimum SRT of 12 days in all four reactors, while varying the HRT between 4 to 10 hours (**Table 6**). The organic removals were relatively unaffected by different HRT's. The average BOD_5 removal efficiencies ranged from 93 to 96 %, while the average effluent BOD_5 concentrations ranged from 9.0 to 10.1 mgL^{-1}. This indicates that the system can perform well with respect to organic removals at a 4 hour HRT. However, the trade-off between aeration basin volume and MLSS concentration must be kept in mind.

The relationship between the various parameters being discussed is as follows:

$$F : M = \frac{So}{MLSS * HRT} = \frac{So * Q}{MLSS * V}$$

where, So = influent substrate concentration (mg BOD_5 L^{-1});
Q = flow (Ld^{-1}); and
V = aeration basin volume (L).

Close examination of this equation indicates that when So and Q are fixed, the combination of a high SRT (which corresponds to a low F:M) with a short HRT, results in a high MLSS concentration. The increase in MLSS, as the HRT or aeration basin volume (V) decreased during phase 2, is shown in **Table 7**. The MLSS observed in R1 is higher than the normal range of MLSS concentrations recommended. Even a small outbreak of filamentous bacteria may cause a bulking sludge episode, while at the lower MLSS of R3, a small outbreak can be controlled without a bulking episode. Therefore, the HRT at the 5-year design PWWF should not be less than 6 hours, based on biomass control rather than organic removal efficiency.

Nitrification

Even though the removal of ammonia-nitrogen (NH_3-N) was not a primary objective, it is an important consideration due to the limited receiving capacity of the Assiniboine River. The TKN-N and NH_3-N concentrations of the activated sludge influent are higher than normal domestic wastewater (**Table 3**), therefore, nitrification becomes an even more important concern. The optimum SRT of 12 days will allow the growth of nitrifying bacteria. Therefore, achieving effluent NH_3-N concentrations of less than 1 mgL^{-1} should be possible under the optimum operating conditions identified in the two experimental phases. In fact, during week 27, R2 achieved 98% NH_3-N removal with an effluent NH_3-N concentration of 0.5 mgL^{-1}.

TABLE 6. Operating conditions of the Activated Sludge Reactors during Experimental Phases 1 and 2 of the Pilot Study

	PHASE 1				PHASE 2			
REACTOR	SRT (d)	*F:M COD:MLSS (d^{-1})	HRT (h)	DURATION (d)	SRT (d)	*F:M COD:MLSS (d^{-1})	HRT (h)	*DURATION (d)
1	12	0.14	8	23	12	0.16	4	11
2	8	0.21	8	21	12	0.22	8	11
3	5	0.31	8	16	12	0.23	6	12
4	3	0.37	8	22	12	0.20	10	11

* The F:M was calculated. The SRT was used for biomass control.

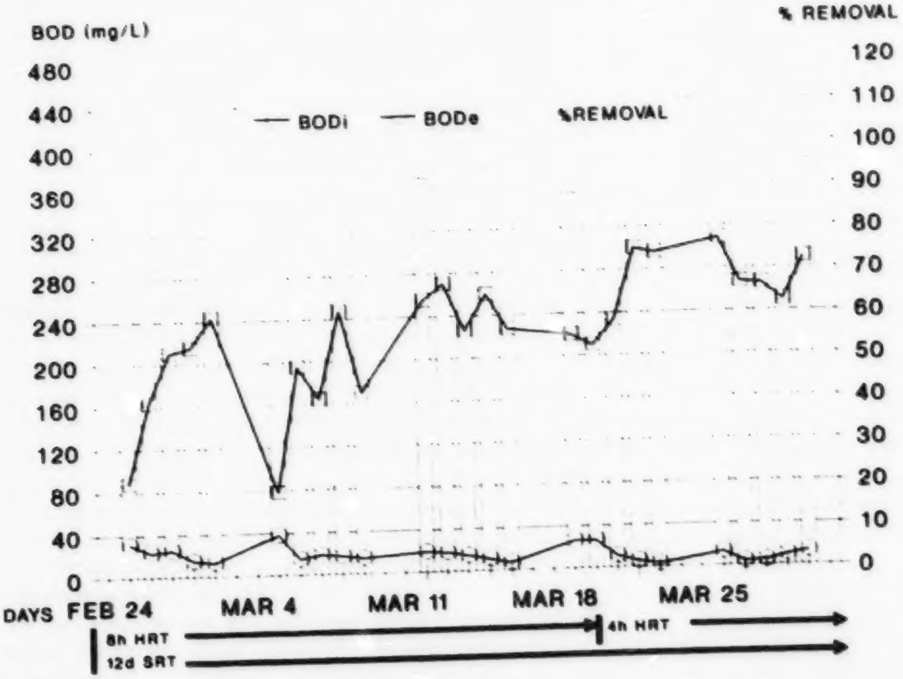

FIGURE 6. Plot of typical BOD_5 data for the activated sludge reactors. The data was taken from reactor and includes the influent BOD_5, (BODi), the effluent BOD_5 (BODe), and the BOD_5 removal efficiences during the pilot study.

TABLE 7. Relationship between the HRT and the MLSS of the Activated Sludge Reactors during the Second Experimental Phase

Reactor	HRT (h)	MLSS (mgL^{-1})
R1	4	5000
R2	8	2500
R3	6	3200
R4	10	2200

Summary

The BVF achieved approximately 80% organic (COD and BOD) removals and 95% SS removals at HRT's of both 10 and 6 days (**Table 4**).

The BVF operated without alkalinity supplements during the final four weeks of the study. Reactor performance remained stable during this time, suggesting that supplements will not be needed after the start-up period. Nutrients will not be required after the start-up period.

WAS loads of up to 10% of the influent COD showed no detrimental effects to the overall BVF process efficiency. This allows for the addition of all WAS to the BVF during the 20 year design period.

The installation of a preaeration facility is recommended, due to the high and variable dissolved sulfide concentration of the BVF effluent.

The effect of, the various activated sludge SRT's and HRT's investigated, on the organic removal performance was essentially negligible. High effluent SS, associated with sludge bulking, was identified as the major issue, therefore it was important to optimize the control of the biomass in the system. The optimum control of the biomass was observed to be at an SRT of 12 days and an HRT of 6 hours in this study.

Effluent ammonia-nitrogen concentrations of less than 1 mgL^{-1} were achieved, at a high SRT (12 days) in the activated sludge reactor.

References

Metcalf and Eddy (1979), Wastewater Engineering, 2nd Ed, McGraw-Hill

NOMENCLATURE AND ABREVIATIONS

AS - Activated Sludge
BOD_5 - 5 - Day Biochemical Oxygen Demand
BVF - Bulk Volume Fermenter
COD - Total Chemical Oxygen Demand
COD_e - Effluent Chemical Oxygen Demand
d - Day
$DBOD_5$ - Dissolved 5 - Day Biochemical Oxygen Demand
DCOD - Dissolved Chemical Oxygen Demand
DO - Dissolved Oxygen
FOG - Fats, Oil and Grease
h - Hour
HRT - Hydraulic Retention Time
L - Litre
mg - Milligram
mgL^{-1} - Milligrams per Litre
ML - Million litres
MLSS - Mixed Liquor Suspended Solids
MLVSS - Mixed Liquor Volatile Suspended Solids
NH_3-N - Ammonia Nitrogen
OLR - Organic Loading Rate
OUR - Oxygen Utilization Rate
PA - Partial Alkalinity, equal to the Total Alkalinity minus the Volatile Acid Alkalinity
Q - Flow
Q_G - Gas Flow
SOC - Soluble Organic Carbon
SRT - Solids Retention Time
SS - Suspended Solids
SS_e - Effluent Suspended Solids
SS_i - Influent Suspended Solids
SSV - Settled Sludge Volume
SVI - Sludge Volume Index
T - Temperature
TA - Total Alkalinity
TKN - Total Kjeldahl Nitrogen, which includes Organic and Ammonia Nitrogen
TOC - Total Organic Carbon
TP - Total Phosphorus
UM - The Environmental Engineering Division of the University of Manitoba
VA - Volatile Acid Alkalinity, the Alkalinity Neutralized by Volatile Acids
VSS - Volatile Suspended Solids
WAS - Waste Activated Sludge
ZSV - Zone Settling Velocity

NOVEL TECHNOLOGIES IN MONITORING BIOFOULING

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INTRODUCTION

Biofouling has become recognized as a major constraining factor in the operation of water related collection, transmission, treatment and delivery systems. This term may be defined as covering any biological event which may occur within the water itself or as an interfacing biofilm between the water and the supporting solid media to grow in such a way as to cause a deterioration in the product water from that system. Such deteriorations resulting from biofouling can take the form of accelerated hygiene risks, sporadic increases in suspended particulates, significant shifts in the chemistry of the water, changes to physical parameters such as pH and redox potential, and the degeneration in the acceptability of the water to the consumer through the generation of chromogenic (colour-forming) and odoriferous compounds. Within the water, the main biological generators of these events would reside in either the interfacing biofilm (which are frequently generated as slime coatings on any of the solid media) or within suspended particles in the water itself. There is a constant interchange between the biofilm and the suspended particles as a result of shearing events in the biofilm which lead to the sloughing and suspension of parts of the biofilm into a particulate phase. Conversely, the suspended particles may become entrapped in the slime interface and again integrated.

A myriad of activities occurs within the biofilm or the suspended particles which will significantly change the quality and deliverable quantity of product water. Quality may change as a result of an increasing microbiological loading because of shearing. This could lead to increasing cloudiness, greater hygiene risk, and changes in some of the chemical parameters. Of these parameters, the most common changes recorded are: reductions in the oxygen concentration, radical fluctuations in the concentrations of iron and manganese, sporadic changes in turbidity, shifts in the redox potential (Eh) and pH, reductions in flow (as the biofilm becomes thicker and more occlusive to the passage of water), increases in the occurrences of corrosion (associated particularly with the generating biofilm after it has matured and stratified), evolution of hydrogen sulphide gas which has been microbiologically generated, and the eventual complete system failure associated with either corrosivity or plugging.

Such a potential for the biofouling3 to cause greater system failures along with a gradually reducing operating efficiency demands that technologies be utilized to monitor the early occurrences of such potentially serious biofouling events. Over the last ten years, there has been developing a greater level of sensitivity in the monitoring of biofouling using both indirect and direct technologies. Before it is possible to understand the applicability of these technologies, it is necessary to comprehend the form and effects that biofouling can have upon water process systems.

Theoretical Factors Influencing Biofouling

Initiation of a biofouling event occurs when organisms enter into a system in such a way that they initiate a chain of events which changes significantly at least one part of the process that is occurring. For example, in a water well, once the borehole has been drilled along with the concurrent disruption to the groundwater media in the immediate zone, there is frequently generated a shift in the environment in that area which will cause greater levels of microbial attachment and growth. Through these activities, the microorganisms may now generate into biofilms which may eventually cause either plugging or radical degeneration in the physical and/or chemical characteristics of the water.

The biofilm forms as a result of microorganisms attaching to solid surfaces, reproducing and colonizing over those surfaces to form a coherent film of growth. Once formed, the biofilm now enters into an expansion growth phase in which a prime objective is for the incumbent microorganisms to generate a sustaining habitat. To do this, the biofilm passes through a number of growth phases which involves: [1] the entrapment of a maximum volume of water [radical swelling]; [2] a secondary compression in which the organisms become more concentrated within the film and at the same time protected by the structures they have created outside of the cells to form the biofilm; [3] a pulse-like secondary increase in the growth of the biofilm while at the same time the microorganisms within the pulsing slime form into stratified layers; [4] periodic shearing of the biofilm leads to releases of particulates which move into the mobile water phase; and [5] the biofilms gradually becomes so large that they interconnect and cause a plugging of the system.

In monitoring the potential risks for biofouling, it is therefore very important to be able to intercede in such events before major irreversible degenerations have occurred. Monitoring technologies are therefore being developed in order to provide "early warning systems" that would allow remedial actions to be undertaken before such irreversible damage has occurred. Methodologies for undertaking these monitoring activities range in type from the early detection of attaching bacteria to the surfaces of solid media, to changes in the particulate loadings in the water and shifts in the microbiological, chemical and physical characteristics of the water 4,5,6. These monitoring procedures are challenged by the constantly changing nature of the biofilms which may be involved in the biofouling. Such biofilms may on one day of sampling prove to be very coherent and appear to have relatively little impact on the quality of the product water, while on a different day, impart very significant changes. The time of sampling therefore becomes critical to the successful diagnosis of a biofouling event. In general, it has been observed that there is a greater potential for the releases of microorganisms from a biofilm while the system is in a quiescent [static] state rather than being actively pumped. When active transmission is not occurring, there generates a shortage of oxygen and nutrients within the system which can cause additional stress to the incumbent microorganisms within the biofilm. One automatic response to these stresses is for the organisms to begin to slough as particulates from the film as sheared particulates into the liquid water phase in preparation of moving to a more facilitative econiche.

In the determination of whether a biofouling event is occurring or not, there is a natural desire to utilize water samples from the possible afflicted zone as a mechanism to obtain the relevant information. Such water would contain the products of any physical and chemical changes to the water along with an incumbent population of the nuisance microorganisms. Technologists who monitor biofouling may therefore either directly determine the microbial populations present, indirectly determine the microfloral population by activities that have been generated, evaluate the changes in concentrations of products of microbial activity or through the variations in the quality and/or quantity of water passaging through the particular process.

The newer technologies being developed to determine the risk of a biofouling event occurring in a water system utilize a variety of strategies. Such technologies range from an evaluation of the amount of total suspended solids (TSS), through to the activity level of specific groups of bacteria[7], to the determination of the attachability of organisms to surfaces which will initiate the biofouling processes. In the earlier examples, it is necessary to obtain samples of water which have transmitted through the process where biofouling may be suspected. It is therefore important to undertake a sampling program that will enable an acquisition of useful data to predict such biofouling.

Sampling Methods for Water in Relation to a Biofouling Event

Biofouling may most easily be recognized when the product water changes in colour, smell or in the deliverable volume. When these events occur, the biofouling event would already have matured and severe process problems would already have been created (e.g., plugging, corrosivity). In any sampling program, it is therefore necessary to be able to determine the risk of a biofouling event occurring before it reaches such a level of severity that the process may be seriously, if not permanently, impaired[8]. This therefore requires that efforts are directed in the sampling program to maximizing the potential for observing biofouling during its early stages of maturation (i.e., attachment, early biofilm formation, swelling and shearing). To achieve this, the sampling program must incorporate a set of samples during which any biofouling process may become magnified by the interceding steps involved. For example, if before taking the water sample, the process is neutralized so that there is no hydraulic movement, then a quiescent (static) phase is established in which an additional stress is placed upon the biofouling microorganisms. These stresses relate to a diminution in oxygen concentration, greater competition for nutrients and degeneration of the environment to a less acceptable state for continued growth. One response of organisms within a biofilm to these traumatic events is to allow a shearing of the biofilm to occur so that some of the bacteria now enter the liquid water phase as suspended particles. These sheared suspendable particles (SSP) will move once water transmission is activated through the water towards potentially more acceptable econiches for colonization. In a sampling program, it is therefore necessary to take sufficient samples to assess whether a quiescent phase has, in fact, caused a greater suspension of microorganisms into the water (Figure One).

A relatively simple scenario can be developed as a means of obtaining suitable water samples. These scenarios all involve sampling the water prior to a period of quiescence, leaving the process or system quiescent for a period of time (e.g., twelve hours, one day, seven days) and then sampling intensely the product water once flow is again initiated. In these latter samples, radical increases in microbial loadings and total suspended particles would be directly indicative of a biofouling event which was traumatized by the period of inactivity.

Water wells are a particular case in point where biofouling may frequently occur in the groundwater formations around the well and not be directly evident within the well itself. To determine where the biofouling was occurring in and around such an installation, a number of sampling programs to determine the biofouling risk had been proposed.

Double Two Sampling Program

The simplest sampling program referred to as the "double two" sampling program involves the sampling of the well at the well head prior to turning the well off for a period of twelve to twenty-four hours. During this time period, stresses will be created in the microflora associated with the biofouling which will cause particulate releases to the water phase. Once the well is activated, two water samples are taken at two minutes and two hours after the initiation of pumping. The first sample (two minutes) includes microorganisms released into the water column, transmission pipes and in the immediate environment around the well screen. The second (two hour) sample includes water which has passaged into the well from the groundwater medium around the well and will include suspended particulates involving biofouling microorganisms where the event was occurring at that point. If no biofouling is occurring, then similar data would be obtained from all three samples. However, where biofouling had occurred to a significant extent, much higher populations of microorganisms and suspended particles would occur in the two minute sample (if the biofouling is close to and within the water well itself) and two hours (where the biofouling was more deeply seated in the aquifer). It is possible to locate the position of the biofouling in and around the water well by undertaking a more extensive evaluation of the particulate mass and biological activity within the water during a more extensive sampling program. Two such programs have been proposed.

Triple Three Sampling Program

Figure 1

Bacterial Populations and Particulate Loadings Released by Continuously Pumping a Newly Developed Water Well

Figure 1a

Bacterial loadings of bacteria being released from a newly developed well QW1 over a continuous pumping period (x axis, mins). Bacterial populations (y axis) are given as colony forming units (counts) per ml. Four different bacterial groups were enumerated: gross aerobic, pseudomonads, iron related bacteria and pseudomonads able to grow on AA agar.

Figure 1b

Relationship between iron related bacteria and the particulate loading during continuous pumping over time (x axis, mins). Iron related bacteria are given (left y axis) as colony forming units (counts) per ml while the particle counting as total suspended solids (TSS, ppm) in the right y axis.

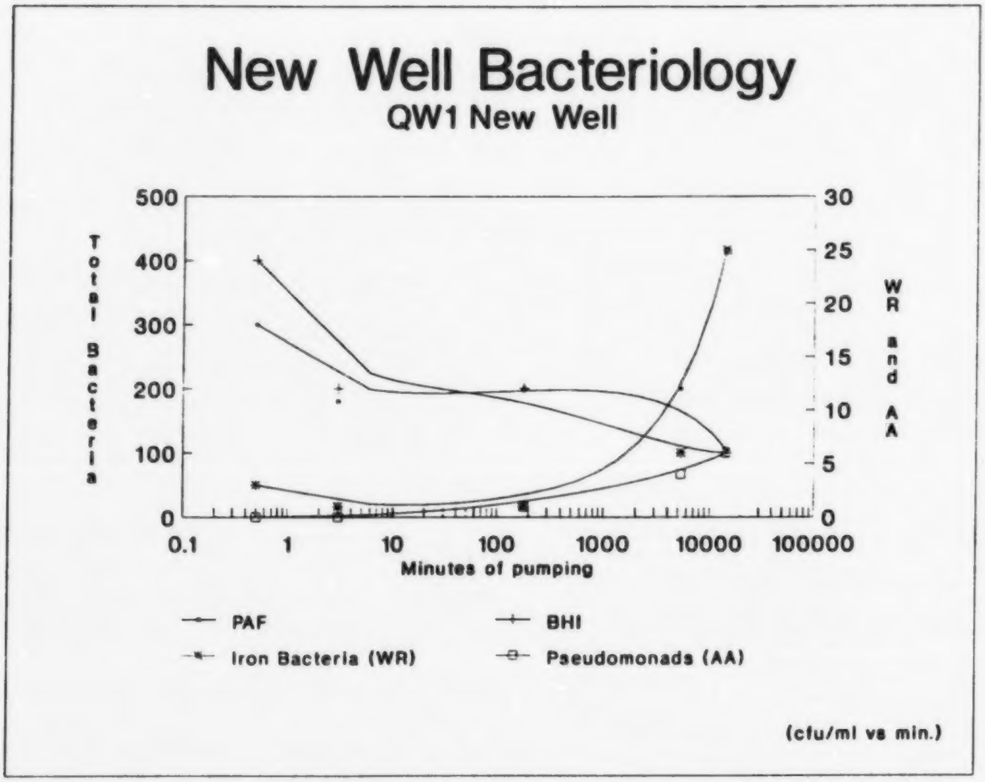


Figure 1a

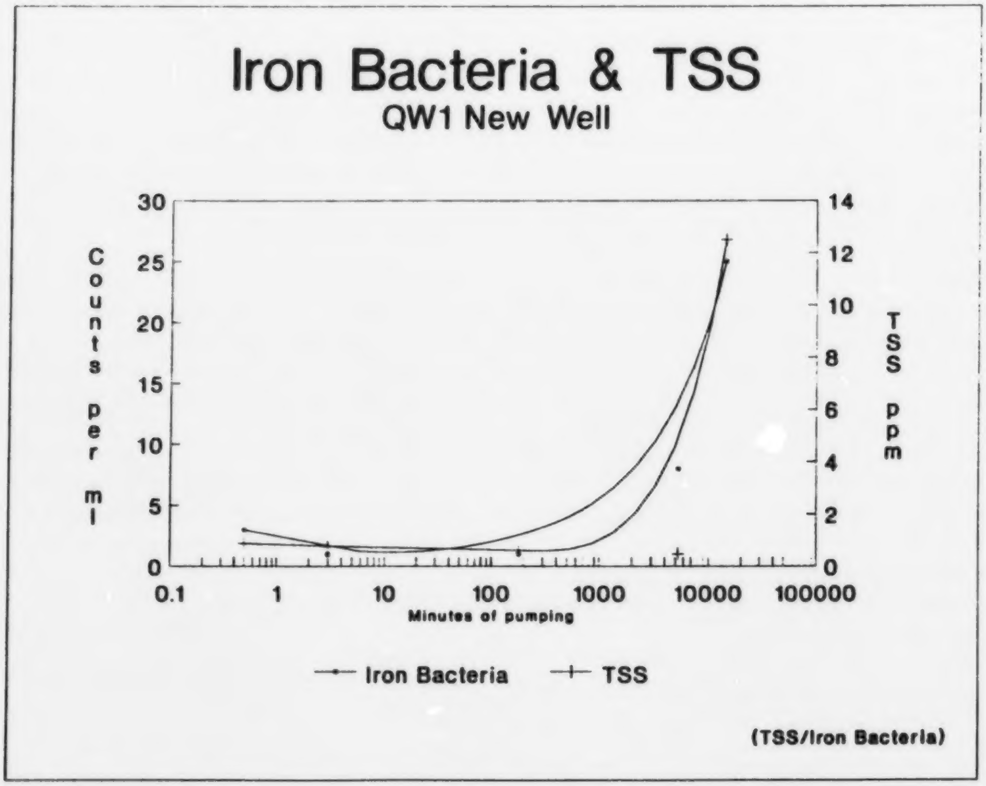


Figure 1b

The next possible sampling program [triple three] involves taking a sample, leaving the well down for a minimum of one day, and then taking three water samples at time intervals of three minutes, three hours and three days after the initiation of continuous pumping after this period of quiescence. If the biofouling is restricted to the well and not particularly matured, then it may be expected that the three day sample would have a similar microbiological and suspended particulate characterization as the original pre-quiescent sample. It should be noted that the well should be continuously pumped for a period of at least seven days prior to the initiation of these sampling procedures.

BAQC Sixteen Sampling Program

A third and more extensive sampling program can be employed to determine more precisely the location of biofouling when it is occurring around the water well as a series of distinctive "cylindrical" growths. This method is called the biofouling assessment quality control [BAQC] sixteen. This methodology involves taking a sample prior to shutting the well down for a period of at least seven days. Once the well is reactivated, samples are taken at one, two, five, ten, twenty, thirty, forty, and sixty minutes followed by two, four, eight, and twenty-four hours and 1.5, 2, 2.5 and 3 days. By comparing the changes in microbiological populations and particulates over this prolonged and intensive period of testing, it becomes possible to extrapolate the size of various cylinders of biofouling which may be occurring around the water well. The technologies for determining the microbial presence has to be sufficiently accurate to allow such biofouling to be determined [Figure Two]. Direct and indirect technologies exist for this determinations. The indirect technology involves the determination of the volume and sizes of the suspended particulates within the water since these would increase where a shearing event had occurred as a result of environmental stress in the biofouling zone [e.g., SSP]. The second technology involves the determination of the activity levels of specific groups of bacteria within the biofouling zone as an index of the risk of biofouling and the aggressiveness of the incumbent bacteria.

Determination of Particulate Loadings in Potentially Biofouled Waters

Traditionally, cloudy water has been thought of as being fouled water and more likely to contain significant microbial populations. In recent years, it has however been discovered that optically clear water with no cloudiness may contain bacterial populations of up to 400,000 cfu/ml. While this would cause concern initially, new technologies involving laser driven particle counters allows these smaller and more covert particles to be not only quantified but also sized and the total incumbent volume of particle material determined. This technology involves the passaging of a rotating beam of laser light through the water in such a way that individual particles interfere with the light

Figure 2

Generation of a Predicted Biofouling Cylinder Around a Well Based upon BAQC Procedures

Figure 2a

Projected dimension of biofouled zones in well ANB by microbiological examination using the BART^{TM} technology. The active well was 61 m in depth and had been newly developed. Four zones (A-D) were defined with the inner zones showing multiple dominance while the outer zone appeared to be dominated by iron related bacteria (reaction 5). The outer edge of zone B was not definable.

Figure 2b

Activity levels of sheathed iron bacteria (SIB-BART^{TM}) and sulphate reducing bacteria (SRB-BART^{TM}) by days of positivity (max. 14, min. 0 - no presence recorded as y axis) during pumping defines zones B and C as the sheathed iron bacteria are replaced by SRB bacteria in the pumped water.

Figure 2c

Zones are more clearly defined by the reaction patterns (numbers, y axis) observed during pumping.

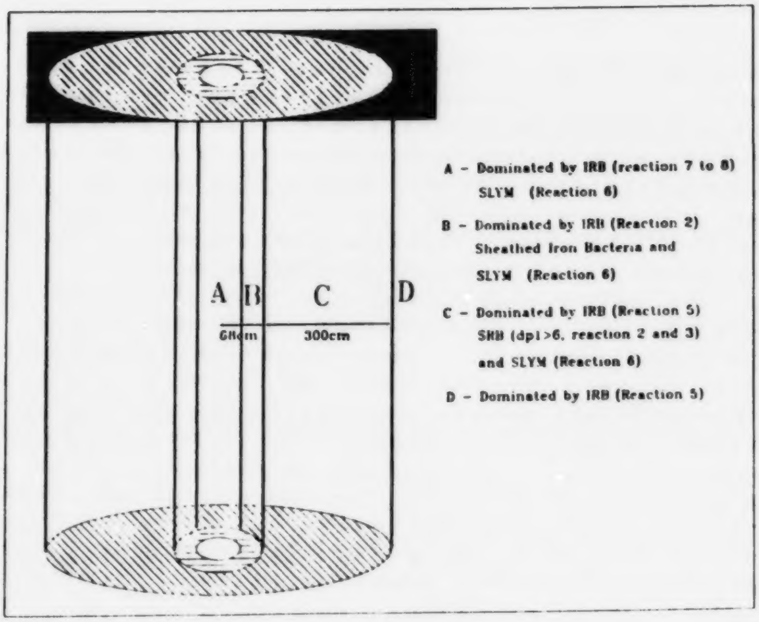

A
B
C
D
6Hcm
300cm
A - Dominated by IRB (reaction 7 to 8)
SLYM (Reaction 6)
B - Dominated by IRB (Reaction 2)
Sheathed Iron Bacteria and
SLYM (Reaction 6)
C - Dominated by IRB (Reaction 5)
SRB (dp1>6, reaction 2 and 3)
and SLYM (Reaction 6)
D - Dominated by IRB (Reaction 5)

Figure 2a

Figure 2b

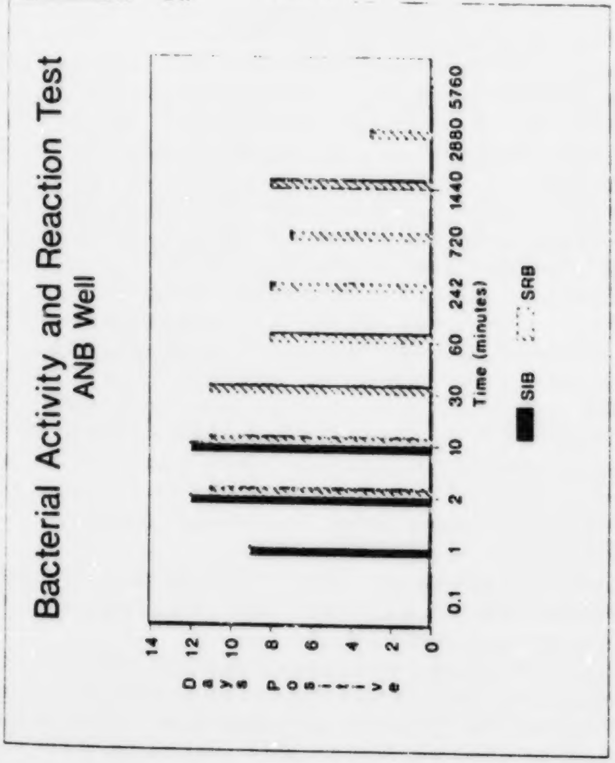
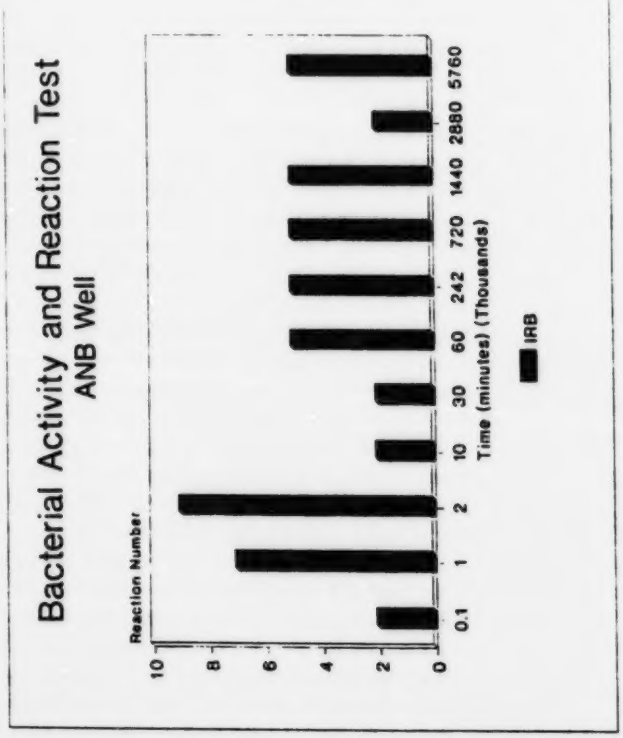


Figure 2c



pulses and are hence recorded so that size and shape of the particle can be determined. In the more recent units, the total particulate volume can be recorded as the total suspended solids [TSS in ppm] and the individual particles sized and separated into a series of particulate bins. These bins are used to determine the number of particles within each of the different discreet sizes such as 8 to 16 microns, 16 to 32 microns and 32 to 64 microns. In routine studies where biofouling events has been observed, radical increases in particulate loadings within the 32 to 128 micron particle size range has often been observed. When employing the sample programs described above, radical increases in TSS with significant increases in the volume distributed in the 32 to 128 micron particle size categories would suggest that a shearing event had occurred during the quiescent phase of the sampling procedure and that biofouling was occurring. In severe cases, the TSS values can reach higher than 50 ppm with greater than 70% of the particulates larger than 32 microns.

Microbial Evaluation of Biofouling Risks

One of the major "stumbling blocks" in the determination of biofouling are the relatively tedious technologies that are used to enumerate the bacterial loadings in water. These involve the use of agar media which are generally applied in the laboratory by trained technicians. The turnaround time to achieve the data is lengthy [e.g., two to three weeks] and frequently, due to pressures to obtain data quickly, the tests are incubated for relatively short periods of time which do not allow the biofouling organisms enough time to grow [e.g., two, three or five day incubation periods]. In many cases, sizeable increases in bacterial populations may be recorded by allowing prolonged incubation periods of up to thirty days. One potential technology to allow the user to more directly determine the amount of bacterial activity in a given water sample is the biological activity reaction test systems [patented BARTTM, Droycon Bioconcepts Inc., Regina, Saskatchewan]. This technology allows specific groups of bacteria to grow within the test by modifying the selective culture medium employed in the tube. The activity of aerobic [oxygen requiring] bacteria and the anaerobic [oxygen avoiding] bacteria is achieved by the use of a floating intercedent device [FID] on the surface of the water. The aerobic bacteria grow at the FID interface and prevent oxygen diffusing down into the lower zones where anaerobic bacteria may now flourish. By modifying the selective culture medium, tests can be performed for the iron related bacteria [IRB-]; sulphate reducing bacteria [SRB-]; and slime forming [SLYM-].

Tests are conveniently established be adding the sample water carefully to the culture tube so that the FID raises to the predesignated mark which is equivalent to 15 ml of water being admitted. The tubes are not shaken and the nutrients in the selective medium gradually diffuse up into the water while oxygen diffuses down from the FID above. At the various interfaces created, different microbial activities occur and reflect the type of organisms present. For example, in the IRB-BART TM there are nine different reaction patterns which reflect the types of organisms present. Experience to date indicates, for example, that a clear green reaction [listed as reaction 8] is associated with the presence of populations of pseudomonad bacteria while reaction 7 is more commonly found where there is a significant population of enteric bacteria.

The semi-quantitative nature of the BART test is generated by the time that it takes for these reactions to occur. Where there are very large and/or aggressive bacterial populations in the water, reactions may occur very quickly. In field experiences with heavily biofouled wells, reaction 8 has been recorded in the IRB-BART TM in times as short as 20 minutes. In comparative trials, it has been found that where the BARTTM test does react to the presence of a bacterial population in less than five days where there is commonly greater than 500 colony forming units [cfu] of bacteria per ml in the water. When there is a six day or greater delay, then commonly the bacterial population is smaller than 500 cfu/ml for that particular group of bacteria. The number of days of delay [dd] has been used to generate semi-quantitative information on the bacterial population. If, for example, during a triple three sampling procedure for biofouling the dd shifted from five days [pre-quiescent] to one dd [three minute sample] to two dd [three hour sample] to five dd [three day sample], then this would indicate that the period of quiescence had caused as significant level of sloughing within the well and surrounding groundwater [i.e., shorter dd values] media and would indicate that a biofouling event was indeed occurring.

In determining the BART data, the days of delay [dd] and the reaction number [Rx] can be used to provide data on the relative activity and composition of the aquatic microflora at that time 7 .

Because the water consists of a number of groups of bacteria which are in competition with each other for nutrients, oxygen, and space, this rivalry continues into the BARTTM test. Very often one group of bacteria which generate one reaction type may be superseded by another causing the reaction pattern in the test to change. This provides an additional identification marker for the particular combination of microorganisms associated with a biofouling event. In some BART reactions observed for periods of twenty days, as many as four distinctive reaction patterns have been observed. These reaction patterns are common to duplicates of the same water sample but do shift where water samples are taken in a chronological sequence of sampling. This therefore acts as a secondary qualitative mechanism for determining the types of microorganisms that are dominant in the water at those particular times. These delays in reaction shifts require that the BART TM be observed daily for longer periods of time. In general, these observations are done conveniently at room temperature and recorded on a daily basis as convenient. The length of time that observations should be performed is fifteen days. However, if no reactivity is observed at fifteen days, it is advisable to continue observations to thirty days in case the bacteria were in severe stress and required an additional time frame in which to recover.

GENERAL DISCUSSION

The covert and insidious nature of biofouling prevents its easy and convenient observation. Technologies9,10 today are falling into place which would allow for the routine monitoring of biofouling in a manner that can be recorded in a quantifiable manner so that management procedures may be introduced to control these events in an acceptable extent. Technologies introduced reflect improved sampling procedures, more sophisticated monitoring of the water for suspended particles, a convenient determination of the biological loading. From this information, the advent of biofouling can be both predicted and managed.

Application of these technologies to a simple set of samples based around a quiescent phase within the water system to be monitored can be used to magnify the biofouling event so that it becomes recordable at earlier stages in its maturation. For large municipal water wells, the BAQC 16 technology would not only provide detailed information on the degree and sites of biofouling around the well but could be used as a monitoring tool in the determination of the rate of expansion of that event through ongoing monitoring.

There has, over a number of years, been an increasing need to more precisely identify the active bacterial components in biofouling events. The simplified BART TM technology allows the user a direct access to methodologies for routinely monitoring given bacterial groups for presence/absence and activity levels in a given water system. Decreases in the days of delay prior to reaction (dd) and the type of reaction obtained can both be used to log in more precise terms the activity levels for these specific groups of bacteria. Management structures can then be used to develop control mechanisms to prevent the biofouling from becoming economically hygienically or politically unacceptable. Management structures may include suppression of the biofouling by the application of such chemicals as hypochlorites11 or inorganic acids, the removal of biofouling by techniques such as the blended chemical heat treatment (patented BCHT process)12,13, or the relocation of the biofouling by manipulation of the environment through the modification of the redox potential by such processes as reinjection of aerated water. It is to be hoped within the next five years that a complete program can be developed through which the biofouling occurring within a municipal or industrial water system can be monitored and routinely controlled by the application of satisfactory therapies.

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Biological Phosphorus Removal
at Lakeview WPCP

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A full-scale demonstration study on biological phosphorus (bio-P) removal was conducted at the Lakeview Water Pollution Plant WPCP) in Mississauga, Ontario. The objectives of the study were to examine the feasibility of bio-P removal at the plant, assess the potential benefits of bio-P removal, and estimate a payback period for a bio-P removal retrofit of the entire plant. The results of the study indicated that a bio-P removal retrofit was possible at the plant. The potential benefits of a bio-P retrofit include an estimated 16% reduction in aeration power requirements, 70% reduction in chemical use, and 17% reduction in sludge production. The estimated payback period for a bio-P removal retrofit of the entire plant is 2.5 years.

Introduction

Phosphorus removal is generally achieved in municipal wastewater treatment plants through physicochemical precipitation with metal salts, such as alum, ferric chloride and lime. This method has proved reliable in reducing plant effluent total phosphorus concentrations to 1 mg/L and less.
However, as effluent requirements become more stringent, the costs for chemical precipitants increase dramatically.
The phenomenon of biological phosphorus (bio-P) removal by activated sludge in excess of that required for normal cell metabolism was first reported in 1955[1]. Since that time much progress has been made in process mechanisms[2-11]. Design and operational parameters have been developed and full scale retrofit evaluations conducted in [12-13].

Several full-scale studies have been conducted on existing municipal wastewater activated sludge treatment plants to determine the feasibility of retrofitting them for bio-P removal[12,13]. Each retrofit situation, however, is site-specific and a plant should be given individual consideration in terms of wastewater characteristics, operational difficulty, and cost effectiveness before a retrofit is implemented.

The Lakeview Water Pollution Control Plant (WPCP) in Mississauga, Ontario currently achieves phosphorus removal by precipitation with ferrous chloride. An assessment of the wastewater to the plant indicated that it met the requirements for a bio-P removal retrofit[7]. On this basis, the plant was chosen as the site for a full-scale demonstration study.

Objectives

The objectives of this study were as follows:

i) to examine the feasibility of bio-P removal at the Lakeview WPCP;

ii) to assess the potential benefits of bio-P removal; and

iii) to estimate a payback period for a bio-P removal of retrofit of the entire plant.

Materials and Methods

Lakeview WPCP

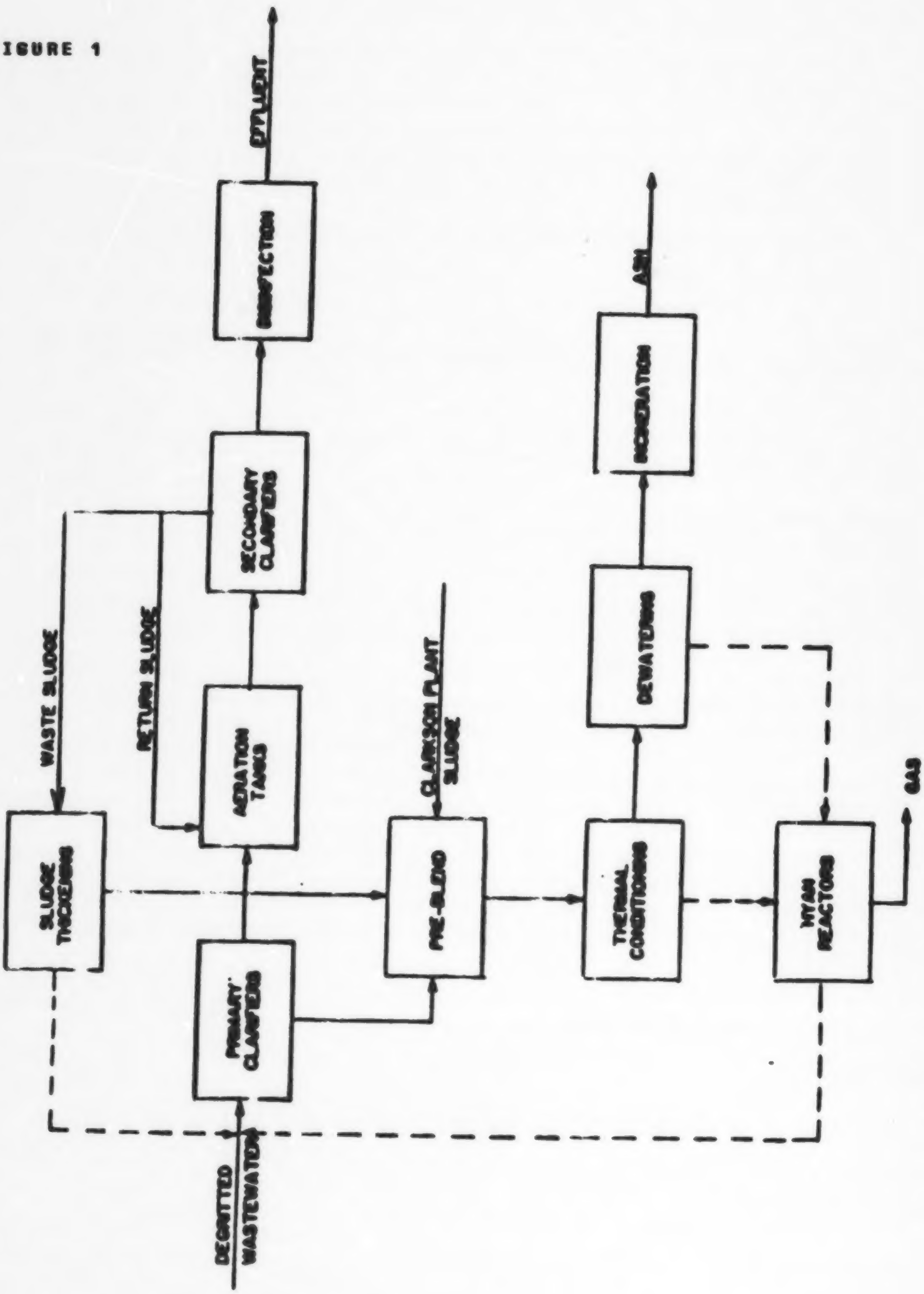
The Lakeview WPCP is located in the southeast section of the City of Mississauga, Ontario. The plant was constructed in 1957 with a design capacity of 22,500 m^3/d. An extension in 1967 increased the capacity to 56,000 m^3/d. A second plant was added in 1972 which increased the capacity to 168,000 m^3/d. In 1975, a third plant was added which brought the total facility capacity up to its present value of 280,000 m^3/d. The plant is presently being expanded to 341,000 m^3/d.
The wastewater undergoes primary and secondary treatment including screening and grit removal, primary settling, aeration, final settling, chemical phosphorus removal and disinfection prior to discharge to Lake Ontario. The secondary sludge undergoes thickening by centrifuge after which it is combined with the primary sludge and sludge from the Clarkson WPCP. The sludge is subject to thermal conditioning and dewatering prior to incineration. Sidestreams from the thermal conditioning unit and the dewatering facility are anaerobically treated in Hyan reactors and recycled to the plant headworks along with centrate from the centrifuges sludge thickeners. Figure 1 provides a process schematic of the plant.

Phosphorus removal is currently achieved at the Lakeview WPCP by precipitation with ferrous chloride. A ferrous chloride solution is added to the activated sludge secondary treatment sections to meet the effluent requirement of 1 mg/L or less of total phosphorus.
The aeration tanks are spiral flow, three pass tanks with diffusers located along one side. During normal operation, the air supply to the tanks is capable of creating a spiral flow velocity of 1.2 to 1.8 m/s at the tank floor.

Modifications for Bio-P Removal

The bio-P study was conducted in Plant 1 and Extension 1. Plant 1 consists of two equally sized aeration tanks [Number 1 and 2] and four final clarifiers [Final 1 to 4]. Extension 1 consists of two equally sized aeration tanks [Number 3 and 4] and two final clarifiers [Final 5 and 6]. both plants receive primary effluent from a common channel. Each plant has a separate return sludge facility. The basic design parameters of Plant 1 and Extension 1 are shown in Table 1.

FIGURE 1


DEGRITTED WASTEWATER
PRIMARY CLARIFIERS
AERATION TANKS
SECONDARY CLARIFIERS
DISINFECTION
EFFLUENT
RETURN SLUDGE
WASTE SLUDGE
SLUDGE THICKENING
CLARKSON PLANT SLUDGE
PRE-BLEND
THERMAL CONDITIONING
DEWATERING
INCINERATION
ASH
WYAN REACTORS
GAS

PROCESS FLOW SCHEMATIC
LAKEVIEW WPCP

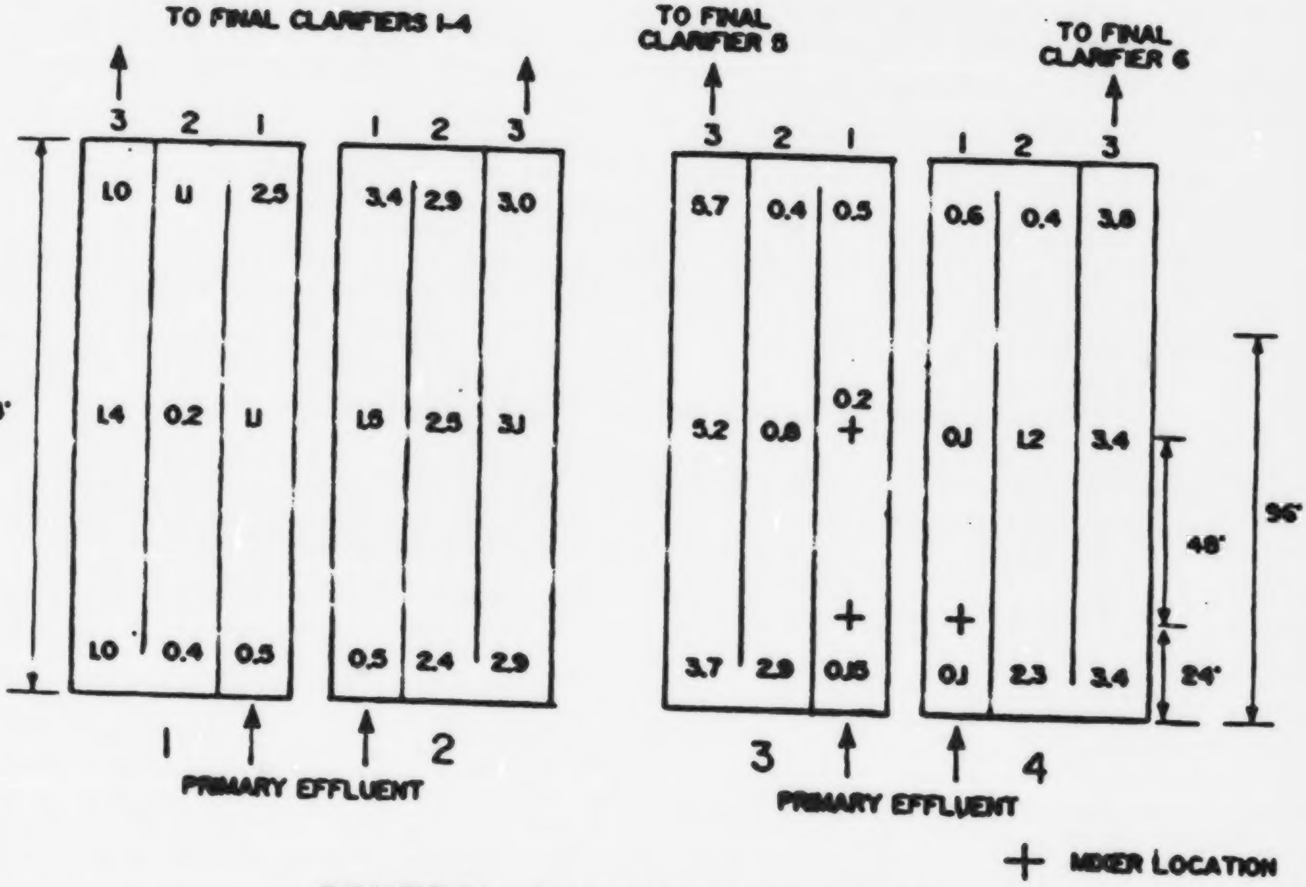
Plant 1 was operated normally as a control system and Extension 1 was used as a test system for the bio-P study. The capacity of each plant is 22 , 500 m^3/d, or about 10% of the total plant capacity. Had the bio-P system failed during the study, the impact on the overall Lakeview WPCP effluent would, therefore, have been limited.

Table 1: Design Parameters of Plant 1 and Extension 1

Component	Design Parameters Plant 1 [Control]	 Extension [Test]
Aeration Tanks		
Number of tanks	2	2
Total tank volume [m^3]	7,200	6,900
Final Clarifiers		
Number of tanks	4	2
Tank area [m^2]	1,126	966
Tank volume [m^3]	3,770	3,540
Weir length [m]	284	338

An anaerobic zone was introduced in the first pass of the aeration tanks of Extension 1 by shutting off the air supply. Mixing was maintained in the tanks through the installation of three 5.6 kW mixers. Due to the fact that demonstration systems were operated with different configurations. Aeration Tank 3 had two mixers in the first two-thirds of the first pass while Aeration Tank 4 had one mixer placed in the first one-third of the first pass. The aeration was reduced to provide a liquid velocity of approximately 0.3 m/s in the next one- third to prevent any solids from settling. The location of the mixers and the dissolved oxygen levels at various points along the aeration tanks used in the study can be seen in Figure 2.

FIGURE 2


TO FINAL CLARIFIERS 1-4
3
2
1
1
2
3
1.0
1.1
2.5
3.4
2.9
3.0
144'
1.4
0.2
1.1
1.6
2.5
3.1
1.0
0.4
0.5
0.5
2.4
2.9
1
2
PRIMARY EFFLUENT
TO FINAL CLARIFIER 5
TO FINAL CLARIFIER 6
3
2
1
1
2
3
5.7
0.4
0.5
0.6
0.4
3.8
0.2
5.2
0.8
0.1
1.2
3.4
3.7
2.9
0.15
0.1
2.3
3.4
48'
96'
24'
3
4
PRIMARY EFFLUENT
+ MIXER LOCATION

CONTROL AND TEST SYSTEMS

Once the aeration tanks in Extension 1 were properly configured for bio-P removal, the addition of ferrous chloride was discontinued. Monitoring of the two plants began four days, or approximately one solids retention time after that time.

Monitoring Program

The primary effluent and the effluent from Plant 1 and Extension 1 were monitored for the duration of the study. Operating parameters, such as detention time, F/M ratio, and SVI, were monitored in both plants as part of the regular plant operation. As the operating conditions of the two aeration tanks in Extension 1 were operated with different configurations, they were monitored individually. Samples taken were 24 hour composites.

Results and Discussion

As previously mentioned, Plant 1 was used as a control system and Extension 1 was operated as a test system for the duration of the study. To ensure that both plants were operating under comparable conditions, the operating parameters for the aeration tanks and final clarifiers were monitored. The average values for these parameters during the study are shown in Table 2.

Table 2:
Average Operating Parameters of Plant 1 and Extension 1

Parameter	Plant 1 (control)	Extension 1 (Test)	
	Aeration Tank 1,2	Aer. Tank 3	Aer. Tank 4
Average Flow (m^3/d)	24,740	11,250	12,430
Aeration			
Detention Time (hrs.)	7.16	7.46	6.72
Organic Load (kg BOD/m3.d)	0.64	0.64	0.74
MLSS (mg/L)	3,026	2,729	3,256
F/M	0.25	0.29	0.39
SVI	83.0	82.7	86.2

Final Settling	Final 1-4	Final 5	Final 6
Detention Time [hrs.]	3.85	3.82	3.54
Surface Overflow [$m^3/m^2.d$]	21.91	23.29	25.73
Weir Overflow [$m^3/m.d$]	87.09	68.28	73.48

It should be noted that Aeration Tank 4 was taken out of service for repairs to be effected to Final Clarifier 6 on day 38 of the study. The values presented reflect the performance of the tank until that point in the study.

Throughout the study, the operation of Extension 1 was carefully examined to determine whether the bio-P system exhibited any operational difficulty. After day 36 of the study, it was observed that some odour could be detected in Aeration Tank 3. On day 38, the operating configuration of the Aeration Tank 3 was changed to that of Aeration Tank 4 [one mixer in the first one third of the first pass and reduced air flow to the next one third]. This was the same day that Aeration Tank 4 was taken out of service for repair to Final Clarifier 6.

The sludge volume indexes during the study for the control and test tanks are shown in Figure 3. With the exception of two high values for Aeration Tank 4, the SVI's recorded for the test tanks were generally below 120 and comparable to the control section. The high values were recorded within the first 10 days of the study and can be attributed to a period of acclimation. No problems were encountered with the final clarifier operation of the test tanks in terms of sludge bulking.

The mixed liquor suspended solids shown in Figure 4 are more variable, particularly for the test tanks. However, the average MLSS values and F/M ratios shown in Table 3 indicate that the overall loading on the aeration tanks was similar. The average F/M for aeration tank 4 was slightly higher because it was taken out of service during a high flow period.

Similar levels of treatment were obtained with the control and test tanks. This is illustrated in Figures 5 through 7 which depict the effluent BOD, SS, and P trends for the duration of the study. The average values for these parameters are listed in Table 4. The effluent BOD requirements imposed by the Ontario Ministry of the Environment were exceeded to a small degree by Plant 1 [the control system]. However, the average effluent P concentration for both plants met the Ministry criterion of 1 mg/L total Phosphorus.

FIGURE 3

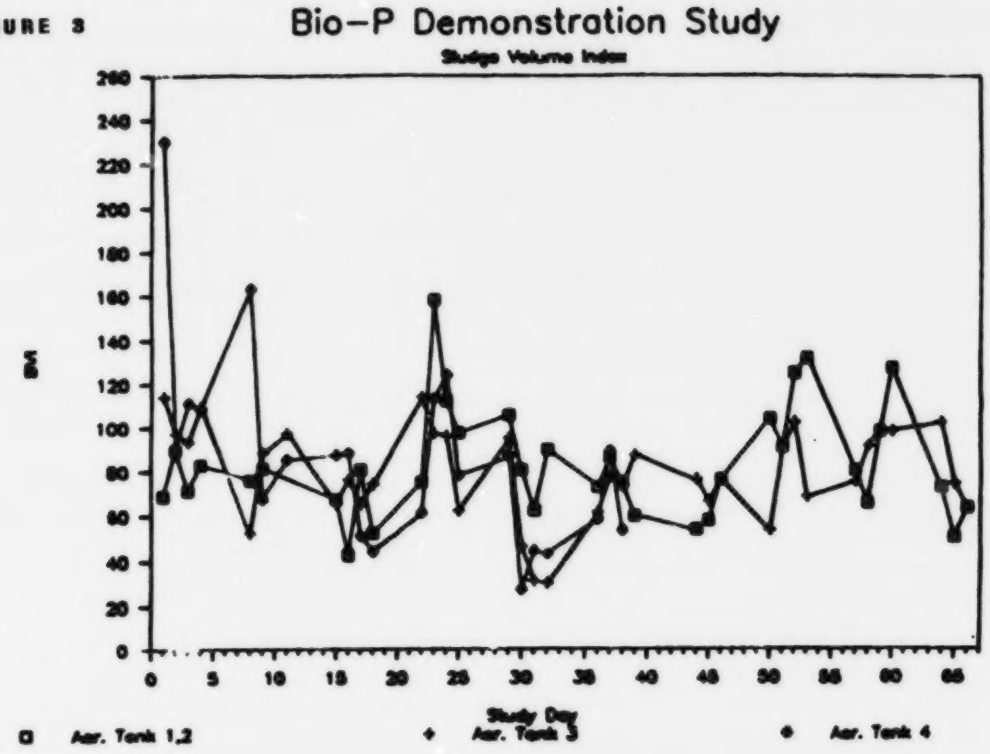


FIGURE 4

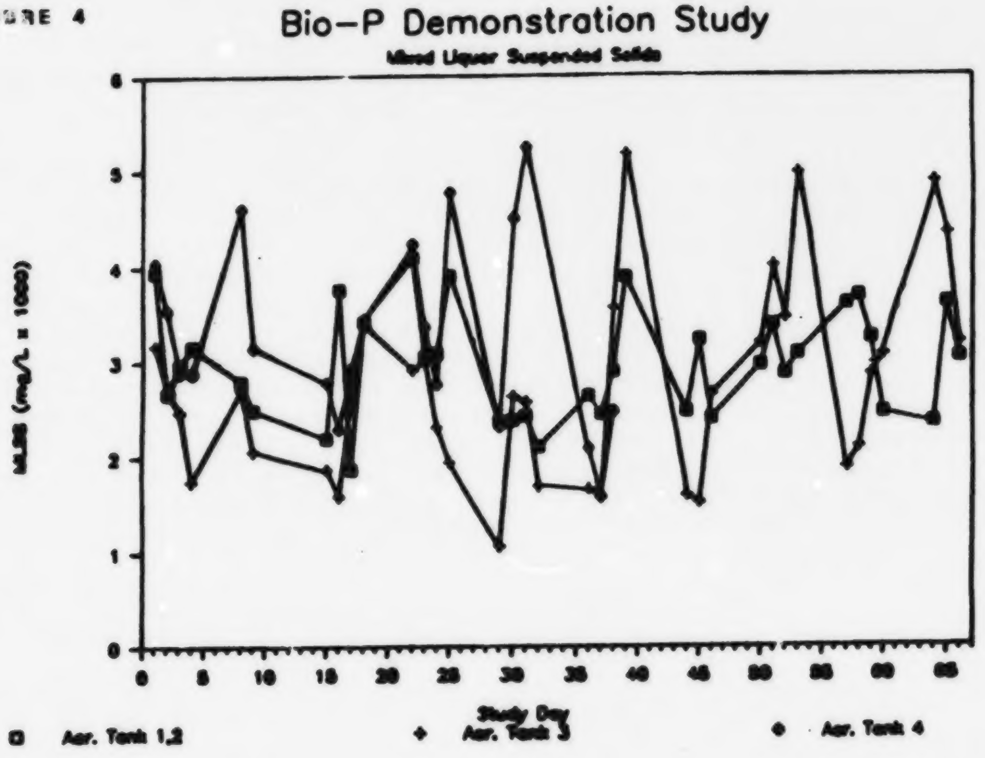


FIGURE 5

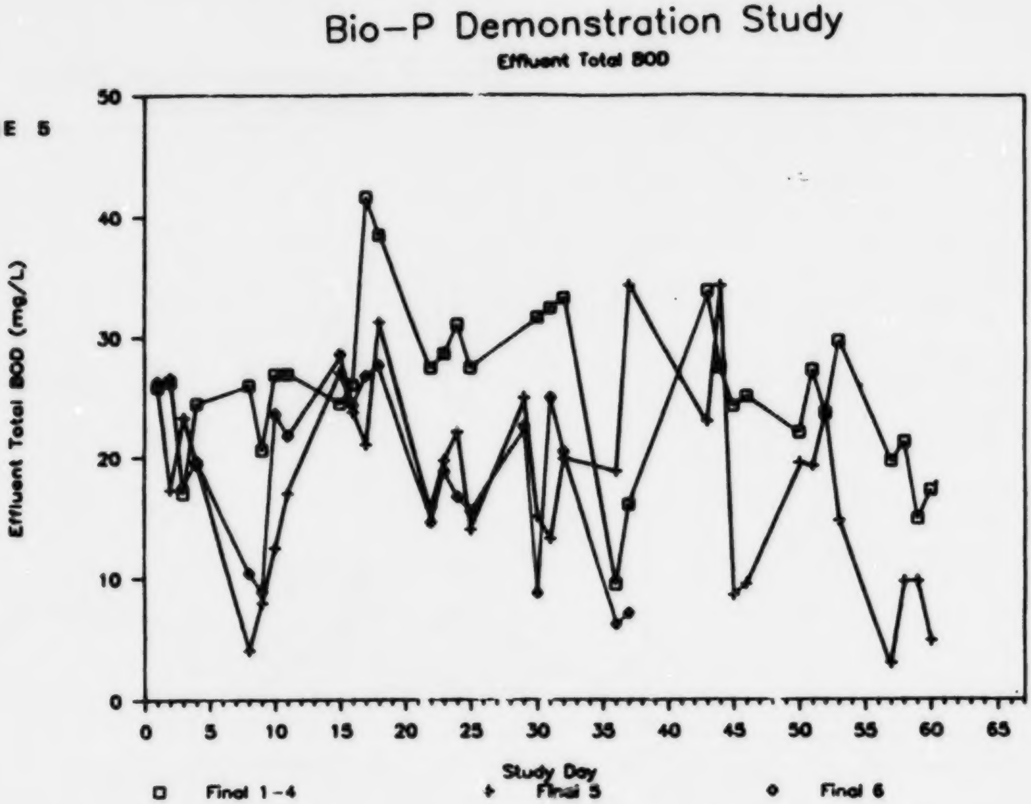


FIGURE 6

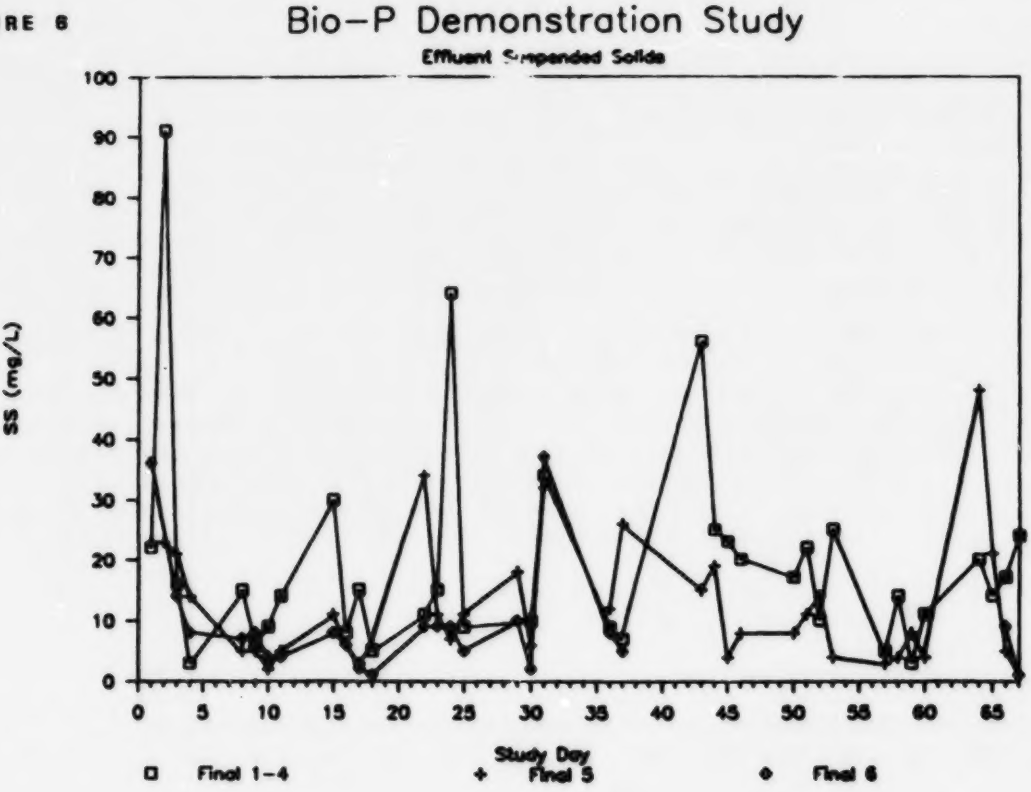


FIGURE 7

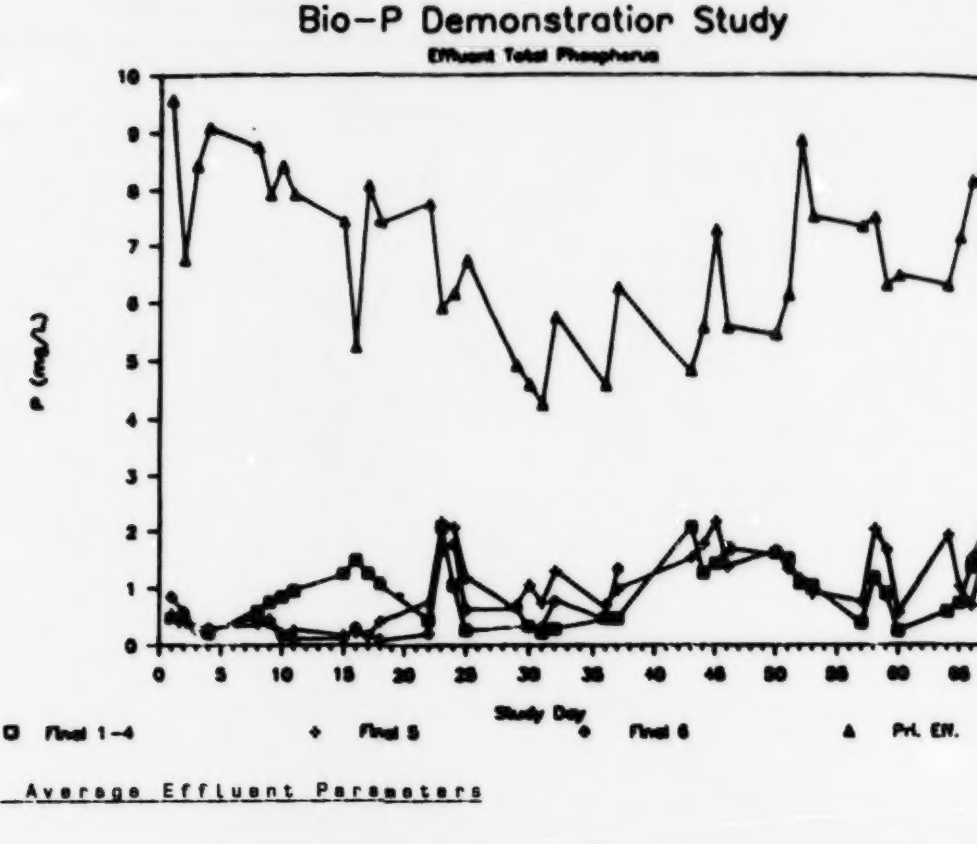


Table 4: Average Effluent Parameters

Parameter	Plant [Control]	Extension 1 [Test]	
	Final 1-4	Final 5	Final 6
BOD [total]	25.2	17.5	17.2
SS	21.5	14.6	23.3
P [total]	0.9	1.0	0.6

The flow and the organic load rate to the control and test systems are shown in Figures 8 and 9 respectively.

Previous research has demonstrated that phosphorus that is precipitated with metal salts is not released under anaerobic conditions[15]. Conversely, phosphorus that has been assimilated for cellular metabolism is released in soluble form. A test program was initiated to determine if the phosphorus in the sludge of the test section was organically or inorganically bound.

FIGURE 8

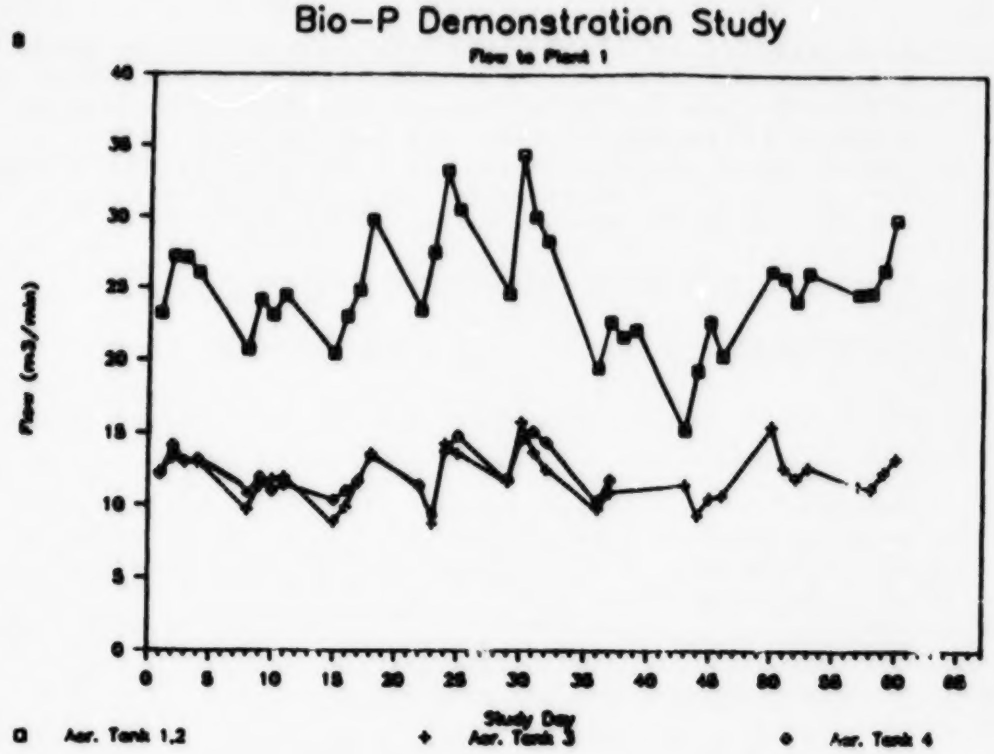
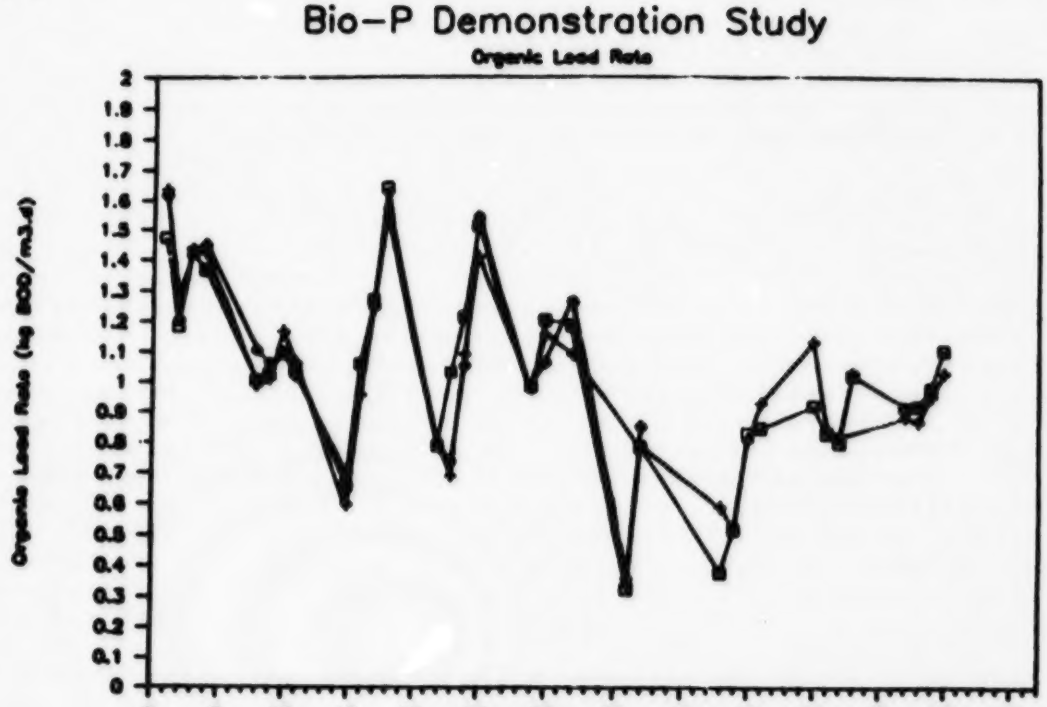


FIGURE 9



Grab samples were taken of the return sludge from Plant 1 and Extension 1. The samples were analyzed for suspended solids, iron, and total and soluble phosphorus. They were then capped and allowed to go septic over a 5 day period. After this time, the samples were once again analyzed for soluble phosphorus to measure the quantity released under anaerobic conditions. The average values for these analyses are listed in Table 6.

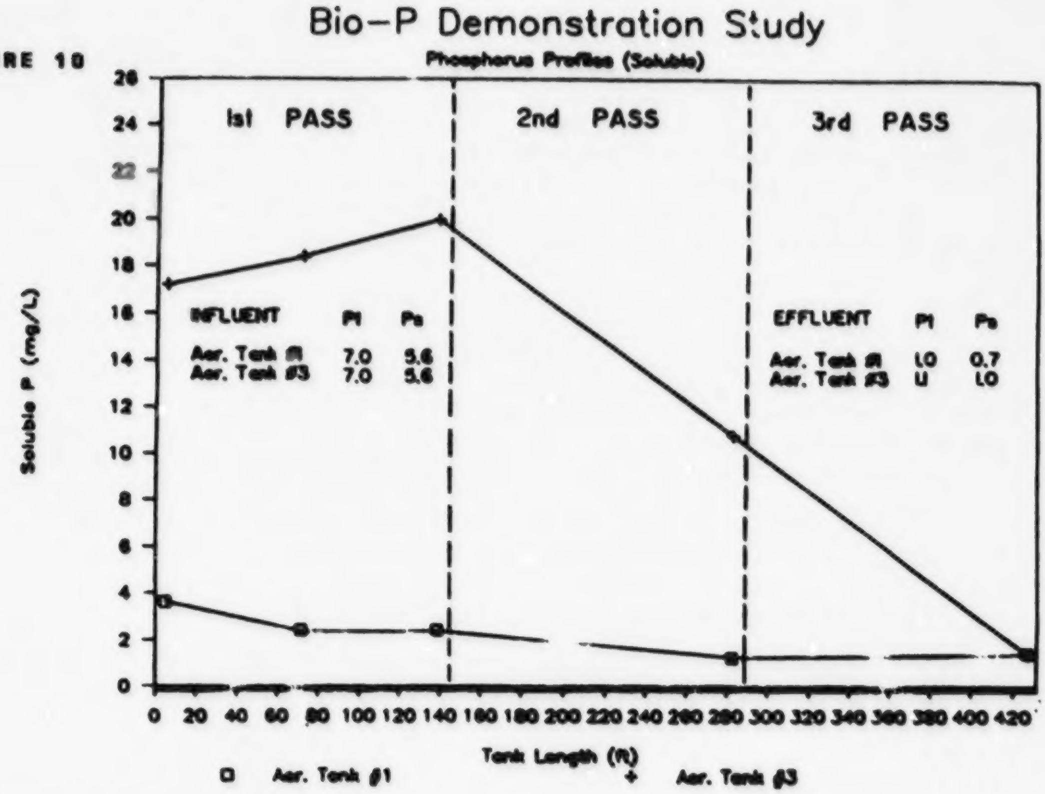
Table 6: Anaerobic Phosphorous Release Tests

Parameter	Plant 1 (Control) Aeration Tanks 1, 2	Extension 1 (Test) Aeration Tanks 3, 4
% VSS	67.2	76.9
VSS (mg/L)	5641 (5943)	4771 (5334)
mg Fe/g VSS	162.6 (155.6)	45.9 (38.4)
mg P/g VSS	47.9 (37.5)	48.0 (35.5)
mg P released/g VSS	0.7 (0.4	6.2 (26.7)
% P released	1.5 (0.9)	14.4 (55.5)

In each case, the amount of soluble phosphorus released by the bio-P system sludge was greater than that released by the control system sludge. On average, this was 14.4% as compared to 1.5%. The numbers in parentheses are average values obtained for two pairs of samples which were allowed to remain under anaerobic conditions for an additional 7 days. For these samples, a much larger average quantity of phosphorus was released by the sludge in the test system 55.5% as compared to 0.9%.

As a further check on the nature of the phosphorus removal in the test system, grab samples of mixed liquor were taken along the length of Aeration Tank 11 and Aeration Tank 3 and analyzed for soluble phosphorus. The resulting phosphorus profile is shown in Figure 10. In accordance with the bio-P removal process described earlier, Aeration Tank 3 exhibited a release of soluble phosphorus in the initial anaerobic section followed by an assimilation of the phosphorus in the aerobic section. In comparison, the soluble phosphorus concentration in Aeration Tank 2 varied only slightly along the length of the tank with reductions occurring in the first section and at the point of chemical application in the end of the second pass.

FIGURE 10



__Benefits of Bio-P Removal__

Substantial costs savings may be realized through the implementation of bio-P removal. The greatest savings are associated with reductions in aeraticn power requirements and chemical precipitant.

There are a total of 18 valves for the control of air to each aeration tank in Plant 1 and Expansion 1. During the bio-P study, 3 of these valves were closed off completely (to provide an anaerobic zone) and another 3 were adjusted to limit the amount of air in each aeration tank (to provide and anoxic zone). On this basis, it was calculated that a 16% reduction in aeration power requirements is possible with the implementation of bio-P removal.

The total cost for electrical power at the Lakeview WPCP is approximately $1,700,000.00 annually, or $4,650.00 per day. An energy audit conducted on the plant revealed that approximately 51%, or $2,370.00 per day is attributed to aeration power requirements. If a 16% reduction in power cost is assumed, then implementation of bio-P will result in a saving of approximately $380.00 per day, or $138,700.00 annually.

Under normal operating conditions, a dose of 10 mg Fe/L in the influent is maintained at the Lakeview WPCP for phosphorous removal. The addition of iron cannot be discontinued completely at the plant because, as with any biological system, upsets may occur.

However, if it is assumed that a concentration of 3 mg Fe/L is sufficient to maintain the required effluent phosphorus concentration of 1 mg/L during adverse conditions (as was demonstrated in a preliminary study), a 70% reduction in chemical use may be realized. In terms of chemical cost, this represents an annual saving of approximately $78,000.00 for the entire Lakeview WPCP (based upon the 1989 cost of chemical precipitant).

An average increase of 35% in sludge volume may be expected in secondary plant which employ metal salts for phosphorus removal[16]. The quantity of sludge wasted from Plant 1 and Expansion 1 was monitored during the study to see if there was a reduction in sludge production through the implementation of bio-P removal. The monitoring program revealed that, on average, 577.7 m^3/d were wasted from the control system as compared to 494.9 m^3/d from the test section, a difference of 17%. Consequently, sludge handling and disposal requirements will be reduced by some 17% through implementation of biological phosphorus removal.

Bio-P Removal Retrofit at Lakeview WPCP

Based upon the successful results of the bio-P study, it was decided to conduct a detailed review of the Lakeview WPCP to assess the feasibility of a bio-P removal retrofit for the entire plant.

The design data for each of the individual plants was tabulated and compared to the design data for Extension 1. Plant operating records for previous years were also examined to determine the most adverse conditions to which the plant would be subjected.

It was found that the design data for the other plants at the facility were comparable to Extension 1 in terms of the parameters critical to bio-P removal. In addition, it was determined that the extreme operating conditions to which the entire plant might be subjected were encountered at various times during the bio-P study as part of the normal plant operations.

From the data obtained during the bio-P study, a capital cost was developed for a retrofit for the entire plant. This was estimated to be approximately $520,000.00. The payback period, for a bio-P retrofit using the annual costs savings presented previously was estimated to be 2.5 years.

Acknowledgement

The authors wish to express their appreciation to the management, operations and laboratory staff of the Lakeview WPCP for their support in this study.

Funding for the initial study, provided by the Research Advisory Committee of the Ministry of the Environment, is gratefully acknowledged.

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THE BUFFALO POUND WATER TRANSMISSION SYSTEM

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Clem Gillete
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Regina, SK

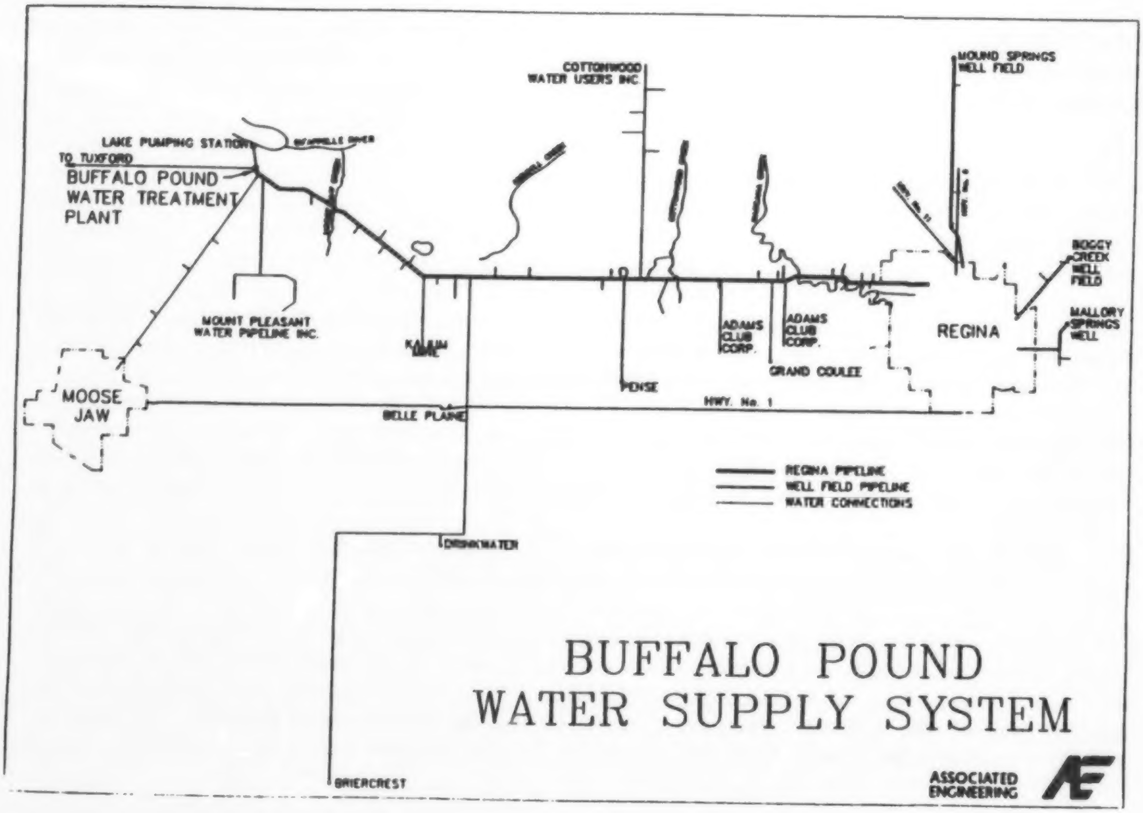
INTRODUCTION

The City of Regina obtains its water from two sources, the Buffalo Pound Water Treatment Plant (BPWTP) and local ground water. The Regina Aquifer supplies approximately 30% of the City's annual consumption.

The BPWTP is located 58 km west of Regina and 19 km north of Moose Jaw as shown on Figure 1. This plant provides treated water to both communities. The BPWTP is operated by the Buffalo Pound Water Administration Board (BPWAB) on behalf of the two cities.

FIGURE 1

Buffalo Pound Water Supply System


LAKE PUMPING STATION
TO TUXFORD
BUFFALO POUND WATER TREATMENT PLANT
COTTONWOOD WATER USERS INC.
MOUND SPRINGS WELL FIELD
MOUNT PLEASANT WATER PIPELINE INC.
KALIUM LAKE
BOGGY CREEK WELL FIELD
MALLORY SPRINGS WELL
ADAMS CLUB CORP.
ADAMS CLUB CORP.
REGINA
GRAND COULEE
PENSE
MOOSE JAW
HWY. No. 1
BELLE PLAINE
HWY. No. 11
REGINA PIPELINE
WELL FIELD PIPELINE
WATER CONNECTIONS
DRINKWATER
BRIERCREST
BUFFALO POUND WATER SUPPLY SYSTEM
ASSOCIATED ENGINEERING

The BPWTP draws water from Buffalo Pound Lake. This shallow lake is located in the Qu'Appelle Valley. Supply to the lake is maintained by controlled releases through the Qu'Appelle Dam from Lake Diefenbaker. This is an abundant supply of excellent water in Lake Diefenbaker.

Treated water is pumped to the City via a system of single and twinned pipelines. Numerous smaller pipelines have been connected to the Buffalo Pound to Regina pipeline. Over 100 farms and eight communities also receive treated water from this pipeline.

HISTORY

In the latter 1940's, water levels in the City's existing wells were falling and the search for additional ground water was unsuccessful. The City began promoting a scheme for using the South Saskatchewan River as an additional source of water.

In 1950, the federal government announced plans to construct a dam at the east end of Buffalo Pound Lake to maintain an adequate supply of water for the Cities of Regina and Moose Jaw. The Buffalo Pound project was eventually constructed for a total costs of $7,000,000.00. This included the intake pipe, pumping station, filtration plant, and pipeline to Regina.

The initial pipeline from the BPTWP to Regina was constructed with much controversy. Tenders were opened in October 1950 for the supply and installation of 57 km of 900 mm diameter pipe. The lowest bid was from the Preload Company of Canada for a prestressed concrete pipeline. This was the latest in technology at that time. However, the contract was awarded to Vancouver Iron Works Ltd. for a steel pipeline even though the price was higher than Preload's. The City engineers did not have confidence in the prestressed concrete pipe.

Steel for the production of pipe became difficult to acquire and as a result Vancouver Iron Works Ltd. surrendered its contract in early 1951. The City then entered into a contract with Preload in July 1951 for the supply and installation of the pipeline.

Production of the prestressed concrete pipe began in February 1952 in a Regina plant. However, production of the pipe was stopped in the fall of 1952 after 30% of the pipe had failed shop and field tests. This case ultimately went to the Supreme Court of Canada and resulted in a judgement in the favour of the City in 1959. The use of calcium chloride as a admixture was generally accepted as the major cause of the pipe failure.

Tenders were again called in November 1953 and the City awarded a contract to Dominion Bridge Co. Ltd. for a steel pipeline. The pipeline was officially placed into service on May 6, 1955. The pipeline has been in continuous and satisfactory service since that time.

The original steel pipeline is 914 mm inside diameter. The pipe is in 12 m (40 feet) lengths connected with compression sleeve couplers. The wall thickness is a minimum of 6.35 mm (0.25 inches) and thicker pipe is used in areas of higher pressure.

The pipeline includes a coal tar lining on the interior and a coal tar kraft paper wrap on the exterior. Both the lining and coating are in very good condition.

The pipeline is cathodically protected against corrosion with an impressed current system. Current readings are made on a monthly basis and adjustments to the rectifiers made as required.

The capacity of the original steel pipeline is limited by the pressure rating of the couplers and valves and not the pipe itself.

The BPWTP originally included a 300 kW (400 hp) and a 112 kW (150 hp) horizontal centrifugal pumping unit for delivering water to the City.

In 1967, a 1500 kW (2000 hp) electrically driven horizontal centrifugal pumping unit was installed. This pumping unit has a variable speed hydraulic drive and operates in a flow range of 70 ML/d (15.5 migd) to 108 ML/d (23.7 migd). This pumping unit was the main duty pump through until the spring of this year when the new vertical turbine pump was added.

In 1975, the Keystown Booster Station was constructed mid way between the BPWTP and the City. The booster station increased the capacity to the City by 25% without exceeding the pressure rating of the original steel pipeline.

By the mid 1980's, water demand at the City had increased to the point where water rationing during peak demand periods was again a reality.

Associated Engineering was commissioned by the City in March 1988 to undertake the engineering necessary for expansion of the water transmission system. The task was to develop and implement a plan for an economical staged expansion of the City's Water transmission system.

ECONOMIC ANALYSIS

An economic analysis was undertaken to determine the diameter of a parallel pipeline which would result in the lowest 50 year present value cost. A spreadsheet was developed on a microcomputer to perform the calculations for the present value analysis. The spreadsheet included technical factors such as the projected growth rate in demand, "C" value of the pipeline, pump efficiency, motor efficiency, and power factor.

Economic factors included capital and replacement costs of the pipeline and pumping equipment, demand and energy rates for electricity, annual maintenance rates for the pipeline and pumping equipment, interest rate, and inflation rate on capital and power costs.

The economic analysis was based on the pipeline twinning being undertaken in four stages with full twinning achieved in the 13th year.

As shown on Figure 2, a 1050 mm diameter pipeline is the most economical over a 50 year life. It is expected that a new steel pipeline will have a useful life in excess of 80 years.

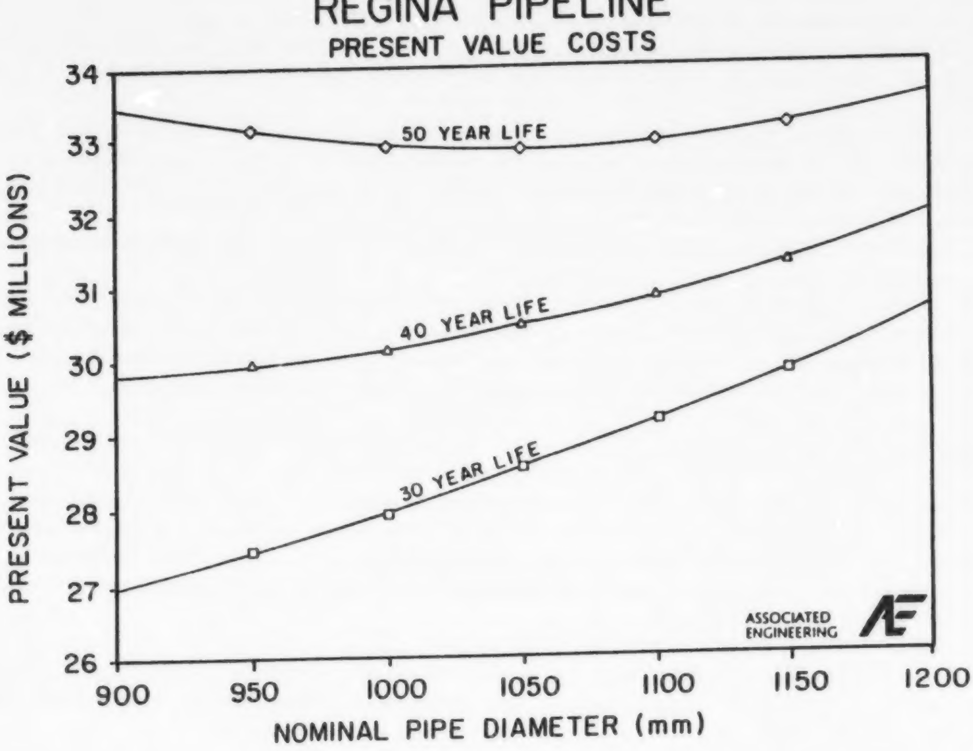
The economic analysis was based on a 2% yearly increase in the peak demand. The growth rate in demand has a significant effect on the economic analysis. The annual power cost is proportional to the yearly water demand to the power of 2.85. A 30% increase in demand will double the power costs.

HYDRAULIC ANALYSIS

In order to undertake the hydraulic analysis, the Hazen-Williams "C" value of the pipeline had to be determined. In the fall of 1988, measurements of the flow rate and pressure drop were taken. A "C" value of 145 was calculated including minor losses through valves and fittings. The results compared well with the "C" value of 146 determined by similar testing in 1982.

The friction loss in metres/kilometers for the pipeline system is shown on Figure 3. At a flow rate of 140 ML/d, the headloss through a section of parallel pipeline (900 mm and 1050 mm diameter) is only 20% of the headloss through the single 900 mm diameter pipeline.

FIGURE 2



The original pipeline is cleaned once a year in the spring to ensure the "C" value remains high. A pig is launched at the BPWTP and forced by water pressure through to a storm channel in northwest Regina. The pigging operation takes approximately 24 hours.

Figure 4 shows profiles of the natural ground, allowable working pressure of the original pipeline and hydraulic grade line for each of the three phases of the pipeline twinning project.

The capacity of the pre-expansion pipeline was limited by two criteria. The suction pressure at the booster station had to be at least 275 kPa (40 psi) to prevent the pumping units from cavitating. The discharge pressure at the booster station had to be limited to 1035 kPa (150 psi) which is the maximum allowable working pressure of the 900 mm diameter pipelines.

The Phase 1 expansion increased the system capacity by 33% while still satisfying the above criteria. Fourteen (14) km of parallel 1050 mm diameter pipeline were constructed immediately upstream of the booster station. This was required to meet the minimum suction pressure at the higher flow rate. Eleven (11) km of parallel pipeline were constructed immediately upstream of the City. This ensured that the maximum allowable discharge pressure at the booster station was not exceeded.

FIGURE 3

REGINA PIPELINE

FRICTION LOSS

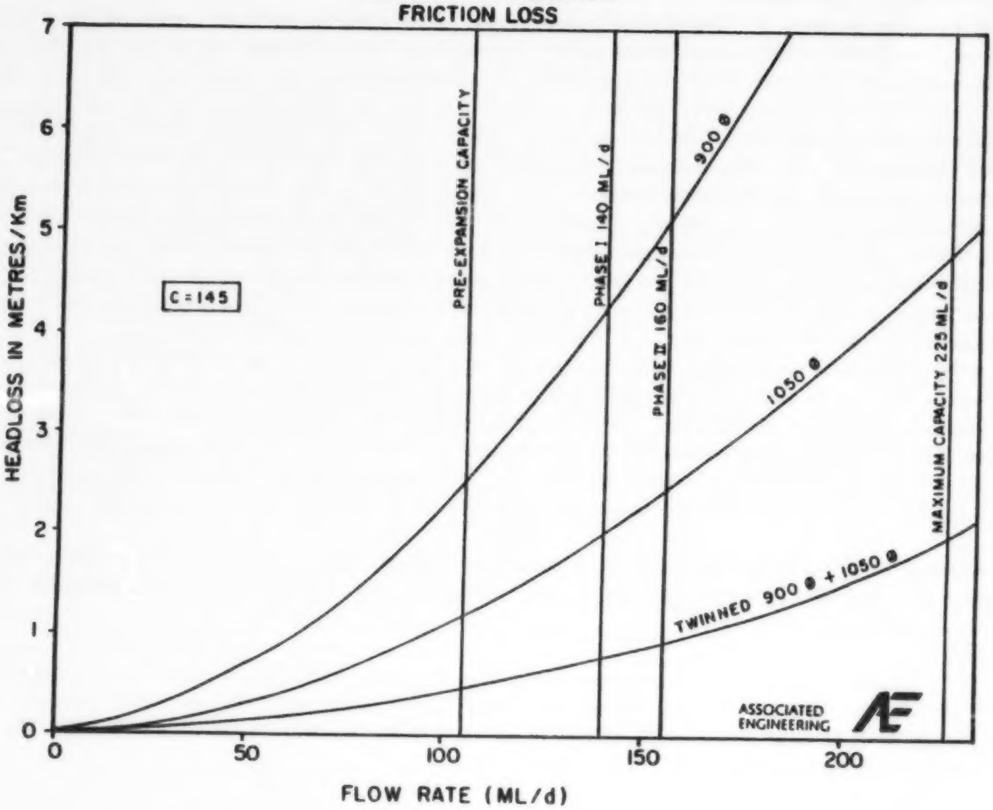
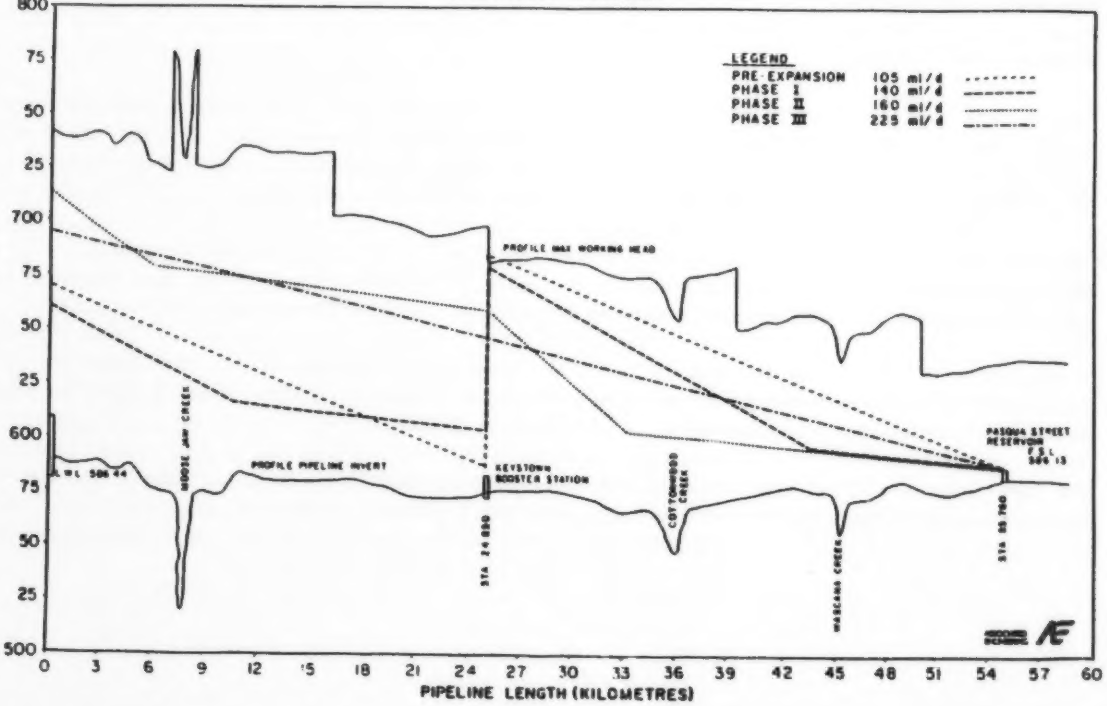


FIGURE 4

REGINA PIPELINE

HYDRAULIC PROFILES



A 1640 kW (2200 hp) vertical turbine pumping unit was added at the Buffalo Pound High Lift Pumping Station as part of the Phase I expansion. This pumping unit was required to meet the increased flow rate. The older 1500 kW (2000 hp) pumping unit provides backup to the new larger pumping unit.

A surge analysis was performed to determine if there was a need for surge control devices. The analysis predicted that an emergency shutdown of the pumping would crease a downsurge in pressure resulting in water column separation in the pipeline. The negative pressure wave could theoretically collapse the pipeline. A surge line was connected between the existing head tanks in the BPWTP and the pump discharge header. On a power failure, the head tanks ensure the pipeline remains under a positive head. A one-way above ground surge tank will also be constructed on the pipeline during the next phase of the expansion. This surge tank will ensure that column separation in the pipeline does not occur during a power failure.

Prior to expansion in 1989, the booster station was operated with three of four pumping units. Field tests conducted in the fall of 1988 confirmed that the station hydraulic performance improved when all four pumps were operated in parallel. In addition, the pumps were operating closer to their best efficiency pcint. The increase in efficiency actually reduced the total operating cost of the four pumps compared to three.

The proposed Phase II expansion will include an additional 15km of parallel pipeline and increase the system capacity of 160 ML/d. The booster station will be abandoned after construction of this phase.

The third and final phase will include an additional 15 km of parallel pipeline and increase the system capacity to 225 ML/d. A second pump similar to the new 1650 Kw (2200 hp) vertical turbine will also be installed in the final phase. It will operate in parallel with the Phase I pump to achieve the required flow rate.

The 1050 mm diameter pipeline is designed for an operating pressure of 1725 Kpa (250 psi). This is significantly higher than the original steel pipeline. In the far future, the City will have the option of isolating the two pipelines and operating the 1050 mm diameter pipeline at higher pressures and flow rates.

PIPELINE CONSTRUCTION

Tenders for the construction of 25.5 km of 1050 mm diameter pipeline were opened in February, 1989. The tenders requested pricing for the supply and/or installation of both steel and concrete steel cylinder pipe. The pipe supply contract was awarded to IPSCO Inc. of Regina. The installation contract was awarded to N.I.S. Contractors Ltd. of Regina.

Construction of the 14 km of parallel pipeline upstream of the booster station began in early May, 1989 and was completed and operational in early July, 1989. The City was able to increase the system capacity by 15% when this section of parallel pipeline became operational.

construction of the 11 km of parallel pipeline immediately upstream of the City began in mid July, 1989 and was completed in late October, 1989. both reaches of the parallel pipeline were successfully pressure tested and accepted in November, 1989.

The steel pipe was lined with two coats of epoxy and coated with two wraps of tape. The lining and coating was factory applied and then the pipe was delivered to the site.

The steel pipe was supplied in random lengths varying from 18 m to 24m long. The pipes were connected with a butt V-joint weld. The contractor welded three of the longer pipes or four of the shorter pipes above ground prior to lowering the assembled section into the trench. Welded connections in the trench were accommodated by excavating a bell hole to allow the welders access all around the connection.

Approximately 40% of the welded joints were radiographically tested. Of those welds tested, approximately 10% had to be repaired. The 483 MPa yield strength of the steel used to produce the pipe required special attention when welding. The joints were preheated to 100°C prior to welding and then wrapped with insulating blankets after the welding to reduce the cooling rate. The contractor also blocked the pipe ends to reduce the air flow through the pipe which was contributing to a very fast cooling rate.

The pipeline was bedded and backfilled with a moderately compacted sand. Measured deflections after installation were found to be less than 2% of the diameter.

Pipeline structures included air valves to allow air in or out of the pipeline as required, blow-offs to allow draining of the pipeline, inter-connections between the two parallel pipelines and line valves for isolating sections of the pipeline.

Fifteen 750 mm diameter gate valves are located on the original 900 mm diameter pipeline. the valve bodies are cast in concrete and cannot be easily removed for repairs. It is known that most of the valves will not fully open. This increases the headloss in the system and causes considerable damage to the pig each year.

When completed the new 1050 mm diameter parallel pipeline will have no more than six line valves. These valves will be full size 1050 mm gate valves. Three valves were installed in 1989.

The only ground water problem which occurred during the 1989 construction was at the Wascana Creek crossing. The site consisted of a saturated sand. Cofferdams were constructed upstream and downstream of the crossing and the creek was temporarily diverted. Shallow berms were constructed adjacent to and parallel with the proposed pipeline. A set of well points were established in each berm. After four days of dewatering the crossing was successfully excavated and the pipeline installed.

HIGH LIFT PUMPING UNIT INSTALLATION

Tenders for the supply and installation of the 1650 kW (2200 hp) were opened in March, 1989. The contract was awarded to Lockerbie and Hole Co. Ltd. of Saskatoon. The pumping unit was manufactured by Ingersoll-Rand and the motor was manufactured by Reliance Electric Ltd.

The pumping unit was successfully installed and commissioned in May, 1990. the pumping unit was a discharge capacity of 140 ML/d which is over 30% greater than the existing 1500 kW (2000 hp) pumping unit.

The new pumping unit is a fixed speed vertical turbine unit. It has a pumping efficiency of 85%. The older 1500 kW (2000 hp) pumping unit has a fixed speed electric motor and a variable speed hydraulic drive. At its design point of 82 ML/d (18 migd), the pumping unit has a factory efficiency of 74%. In February, 1989, field tests were conducted on the pumping ;unit and the pump efficiency was determined at 67%. The new pumping unit will be operated as much as possible because of its significantly higher pumping efficiency.

PUBLIC COMMUNICATIONS

An effort was made to keep the public informed during the design and construction stages.

Separate public meetings were held for the eastern and western reaches of the pipeline. Notices of the meetings were sent to all landowners within four miles of the project. Literature was made available at the door which explained the purpose of the project, provided information on the firms involved on the project, and outlined the proposed construction schedule. The meetings were very informal. This allowed the citizens to talk directly to representatives from the City and Associated Engineering. concerns expressed by the people were documented for follow-up after the meeting.

Signs were erected at each end of the project during the construction period. This made the local public aware of the firms involved on the project incase they had a concern.

Information meetings were held with the two rural municipal councils with jurisdiction in the areas of the pipeline construction. The councils were informed of the construction schedule and given names of key people involved on the project. Issues such as haul routes and the responsibility for the roads were also discussed.

Numerous shutdowns of the old pipeline were required to accommodate construction of the new pipeline. Twenty-four hours notice of impending shutdowns was given to all the existing water users along the pipeline. The existing water users were very tolerant of these shutdowns.

THE YEAR AFTER CONSTRUCTION

The Phase I pipeline expansion has a design capacity of 140 ML/d. field tests were conducted in May, 1990 to confirm the performance of the new transmission system. The new high lift pumping unit produced flow rates of 140 ML/d and 115 ML/d with and without the booster station as predicted.

The suction and discharge pressures at the booster station have been satisfied at the design flow rate with the parallel pipeline in place.

The Booster station has been traditionally operated from early May through to the end of September each year since it was constructed. In 1990, the booster was operated for only 80 hours in the month of June to meet the peak demands. This is primarily due to the expanded transmission system. A significant reduction in operating costs at the booster station was realized this year.

A small number of water users off the pipeline have been adversely affected by the new system. The twinned pipeline has significantly reduced the line pressure in some areas and individual water users are considering adding booster pumps to their water system.

In the twinned pipeline sections of the transmission system, the City now has the option of isolating either pipeline for maintenance purposes while still delivering water to the City. This allowed the City to remove some unused structures off the original pipeline this summer. In addition, modifications were made to the existing piping and structures at Pasqua Street this summer without disrupting the flows to the City.

The pipeline twinning project has been a success in all regards. It has increased the capacity of the water transmission system, reduced operating costs for the City, and provided more flexibility in the operation of the system. The system will be further expanded in coming years as required, to meet the growing demands of the City.

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